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AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

WIND STRESS ANALYSIS SIMPLIFIED

BY L. E. GRINTER,[1] ASSOC. M. AM. SOC. C. E.

SYNOPSIS

One of the most difficult problems that the structural engineer must solve is the analysis of stresses in a continuous frame in which joint displacements occur. In many cases, since the loading is fixed, the analysis does not involve placing loads for maximum stress. The calculation of wind moments in tall building frames is such a problem.

It is the purpose of this paper to present a method, or rather a system of analysis, by which such frames may be analyzed with satisfactory accuracy in a simple manner and in a fraction of the time required by most exact or nearly exact methods. This simplified method of analysis is made dependent upon the system of balancing fixed-end moments introduced[2] by Hardy Cross, M. Am. Soc. C. E. Accordingly, the application of the method requires the use of only the simplest arithmetic. The importance of this fact is great since the designer's mind is left largely free to study the matter of proportioning the structure.

METHODS IN COMMON USE

An exact analysis of a continuous frame, such as a wind bent, has usually required the solution of a group of simultaneous equations, which is a rather tedious process even when simplified to the greatest possible extent. The most usable of these older methods is probably the Slope Deflection Method as advanced by Wilson and Maney.[3] Simplifications of the slope deflection method and of other exact methods have been proposed. Usually, these simplifications have involved the idea of neglecting the effect of moments in one story upon the moments in all other stories except those adjacent to the one under consideration. The approximate methods that resulted were entirely

NOTE.—Discussion on this paper will be closed in April, 1933, *Proceedings*.

[1] Prof., Structural Eng., Agri. and Mech. Coll. of Texas, College Station, Tex.

[2] "Analysis of Continuous Frames by Distributing Fixed-End Moments," *Transactions*, Am. Soc. C. E., Vol. 96 (1932), p. 1.

[3] "Wind Stresses in the Steel Frames of Office Buildings," by W. M. Wilson and G. A. Maney, Members, Am. Soc. C. E., *Bulletin No. 80*, Eng. Experiment Station, Univ. of Illinois, Urbana, Ill.

satisfactory except for the time consumed in the analysis. Particularly in the case of wind stresses in tall building frames the time required for an exact or nearly exact analysis has seemed excessive, and engineers have continued to make wide use of the so-called statically determinate methods. This fact is surprising because these methods have been shown to be in error as much as 100%, even for quite regular buildings.

To date, the most usable method of wind stress analysis, which also can claim to be more than a rough estimate, is that suggested by Ross and Morris.[4] This method is both fast and simple to apply, but gives results that seem to be too greatly in error for careful design. In several cases in the two analyses presented by those writers (the Wilson-Maney bent and the American Insurance Union Building), the moments are shown to vary from the correct values by from 15 to 48 per cent.

The most recently suggested method of analyzing continuous frames is that made by Professor Cross.[2] The Cross method of balancing fixed-end moments is directly applicable to all frames when translation occurs at none of the joints. It is fast and simple, and produces any degree of accuracy desired. The writer considers it to be the most satisfactory method yet proposed for the analysis of frames in which the joints do not move. For cases in which joint translations occur, Professor Cross suggests a method of balancing shears, but this, like the other exact methods of caring for joint movements, requires considerable time. Certainly, for the case of wind stress analysis the time required will be found to be excessive.

General Method of Successive Corrections

The writer has suggested a general method of analyzing continuous frames for fixed loads, which he has been in the habit of calling "Successive Corrections."[5] This method was re-stated and examples of its application to wind stress analysis have been presented by Professor Morris.[6] The first step in the general method of successive corrections is to balance fixed-end moments. A simplification is possible in certain cases and this is the "simplified method of wind stress analysis" suggested in this paper. The simplified method is also applicable to certain structures other than building frames, notably bents, open web trusses, and articulated trusses for secondary stress analysis.

The general method of successive corrections is re-stated herein to form a background for the presentation of the simplified method, as follows:

(*a*) The structure is allowed to deflect under the loads while the joints are fixed against rotation, and the fixed-end moments are calculated.

(*b*) The fixed-end moments are balanced around the joints until each joint is in equilibrium; that is $\Sigma M = 0$ at each joint. During this process no translations of the joints are allowed. This step is the same as in the Cross method of balancing fixed-end moments.

[4] "The Design of Tall Building Frames to Resist Wind," by Albert Ward Ross, Jr., Jun. Am. Soc. C. E., and Clyde T. Morris, M. Am. Soc. C. E., *Proceedings*, Am. Soc. C. E., May, 1928, Papers and Discussions, p. 1595.

[5] *Transactions*, Am. Soc. C. E., Vol. 96 (1932), p. 11.

[6] *Loc. cit.*, p. 66.

(*c*) From the balaneed moments of Step (*b*) the total shears are determined, and from the difference between these shears and the true shears the artificial restraining forces at the joints are found. (There must be restraining forces since the deflections of the structure as produced in Step (*a*) are not the true deflections.)

(*d*) The joint forces found by Step (*c*) are eliminated by applying equal and opposite joint forces.

(*e*) The structure is allowed to deflect with fixed joints under the action of this new force system, fixed-end moments are calculated, joint moments are balanced, and a second set of joint restraints is computed. It will be found that this second set of joint restraints is smaller and of less importance than the first set.

(*f*) The process is repeated until a final set of joint restraints is determined that is small enough to neglect entirely.

(*g*) The final moments are obtained by adding algebraically the successive sets of balanced moments determined from the original applied loads and the several sets of forces that balance the artificial joint restraints.

The foregoing explanation of the full method of successive corrections is given in detail, in order to make the process clear. When stated in this manner, the reader should visualize clearly a physical action of the structure that corresponds to each step of the process. In practical applications the method may be simplified considerably. Since the first balancing of fixed-end moments is disrupted by the application of a second set of fixed-end moments, it is clear that the first balancing need not be carried to its normal conclusion. In fact, it would be a waste of time to balance the individual joints more than twice before releasing joint restraints. Professor Morris[6] has suggested releasing joint restraints after each balancing of the individual joints, which is entirely satisfactory.

Furthermore, there is an obvious simplification in the practical application of Steps (*c*), (*d*), and (*e*), that is, in releasing joint restraints. It is unnecessary actually to calculate the joint restraints at all, because the difference between the sum of all balanced column moments in a given story and the sum of all the original fixed-end moments is equal to the sum of the second set of fixed-end moments to be applied in this story. This total fixed-end moment is to be divided among the columns in proportion to their K-values. Nevertheless, it is of importance that one realize the full significance of this simplification.

The Simplified Method

In the method of successive corrections, the first set of balanced joint moments would have been the true final moments if the first set of artificial joint restraints had been negligible. Moreover, the first set of joint restraints would have been negligible if the deflections of the structure with joints fixed against rotation had been essentially the same as the true deflections. In other words, if the true deflections of the joints can be guessed at, estimated, or actually calculated, it is permissible to force the structure into this deflected position with fixed joints. Then one can calculate the fixed-end moments

caused by these deflections and balance the joint moments. If the estimated deflections are reasonably correct, the joint restraints will be found negligible, and the first set of balanced moments may be taken as the final moments. When the first set of joint restraints is too large to neglect, but is still small in comparison with the actual loads, the moments may be corrected approximately. This is accomplished by assuming that there will be no change in the location of the points of contra-flexure in the members. Hence, the final column moments are obtained simply by multiplying the first balanced moments by the ratio of the true shear to the computed shear in that story. Final girder moments are obtained by dividing the sum of the corrected column moments at each joint between the girders in the same ratio as that of the uncorrected girder moments. An outline of this simplified method is, as follows:

(1) Guess at, estimate, or calculate the displacement of each joint of the frame as caused by the applied loads.

(2) Compute the fixed-end moments in the members as caused by these movements. If the translation is horizontal, there are no fixed-end moments in horizontal members.

(3) Balance the moments in succession at all joints until the joints are fully balanced; that is, until $\Sigma M = 0$ at each joint.

(4) Compute the total shear in each story from the balanced moments. The difference between these shears and the real shears caused by the loads is a fair indication of the error in the moments.

(5) Determine the final column moments by proportion, using the relation:

$$\text{Final moment} = \text{Balanced moment} \times \frac{\text{True shear}}{\text{Calculated shear}}$$

(6) Determine the final girder moments by dividing the sum of the corrected column moments at each joint between the girders in the same proportion as that of the uncorrected girder moments.

When the calculated shear is within 10% of the true shear, one need have no fear but that the final moments will check the true moments within a range of error of about 5% when proportioned as indicated in Steps (5) and (6). This factor of 5% is intended to apply to moments of average intensity. A particularly small moment may be in error by 6 or 7 per cent. In several cases the writer has been able to estimate the deflections of building frames with sufficient accuracy so that the calculated shears have been within 6% of the true shears, and when corrected by proportion the final moments have checked the true moments within 3%, or less. When the frame is irregular, larger errors are naturally to be expected.

The remainder of this paper will be devoted to a development and application of simple methods of estimating the absolute or relative joint deflections of frames before the division of the total moment among the members is known. Relative deflections are as satisfactory as absolute deflections since the final moments are obtained by proportion. Examples of the application of the simplified method to several continuous frames will be given. The

examples chosen have previously been analyzed in the literature of this field so that the percentages of error may be studied. The major emphasis is thrown upon the analysis of wind moments in tall building frames because this is the most important of all problems in the field of continuous frames and the one that has been most neglected.

Re-Statement of the Process of Balancing Moments

In the process of balancing moments, one starts with a set of fixed-end moments in the members. These moments may be caused by intermediate loads or by joint translations. Those caused by intermediate loads are cal-

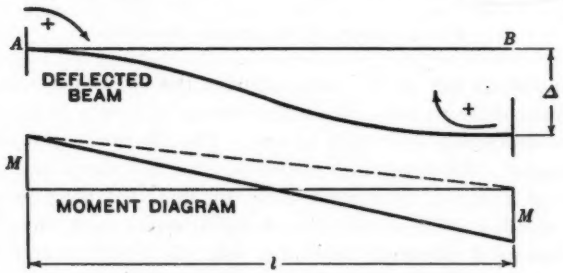


Fig. 1.—Joint Translation.

culated as in any fixed-end beam. When joint translations occur, the fixed-end moments may be computed as follows: In Fig. 1, the deflection at B from a tangent to the elastic curve at A is equal to the statical moment of the $\frac{M}{EI}$ diagram from A to B about B. Therefore,

$$\Delta = \frac{Ml}{2EI} \times \frac{2}{3} l - \frac{|Ml|}{2EI} \times \frac{1}{3} l = \frac{Ml^2}{6EI} = \frac{Ml}{6EK} \quad \ldots\ldots\ldots(1)$$

and the fixed-end moment is,

$$M = \frac{6EK\Delta}{l} \quad \ldots\ldots\ldots(2)$$

The question of signs must be given careful consideration since the sign convention adopted is different from that used by Professor Cross. However, a single statement of this sign convention will cover all cases: When an end moment tends to rotate a joint in a clockwise direction, that moment is always considered positive. In Fig. 1, since the moment in the member, $A\,B$, at A tends to rotate the joint clockwise, this moment is positive; also, the moment at B in the member, $A\,B$, tends to rotate Joint B clockwise, and, therefore, this moment is positive. The deflected shapes of members with the usual conditions of fixed-end moments are shown in Fig. 2. It is possible to find combinations of the cases shown, as, for instance, normal translation plus intermediate loads.

When the fixed-end moments in all members have been determined, the joints are balanced in any desired succession until each joint is eventually freed from any restraining moment. The sum of the moments recorded

around any joint at one time is equal to the total unbalanced moment. The individual moments are in proportion to the K-values of the members meeting at the joint. The balancing moments to be recorded at any one time around a joint are of the same sign and of opposite sign to the unbalanced moment.

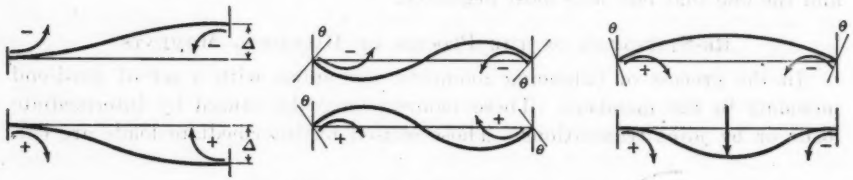


FIG. 2.—SIGNS OF FIXED-END MOMENTS.

Carry-over moments are of the same sign as the balancing moments. They are equal numerically to 50% of the balancing moments if the members are of uniform cross-section from end to end. The algebraic sum of the original fixed-end moment, the balancing moments, and the carry-over moments for any member at a given joint represents the final balanced moment at that end of that member. When this sign convention is used, the algebraic sum of all final balanced moments around a joint is equal to zero; that is, the joint is in static equilibrium.

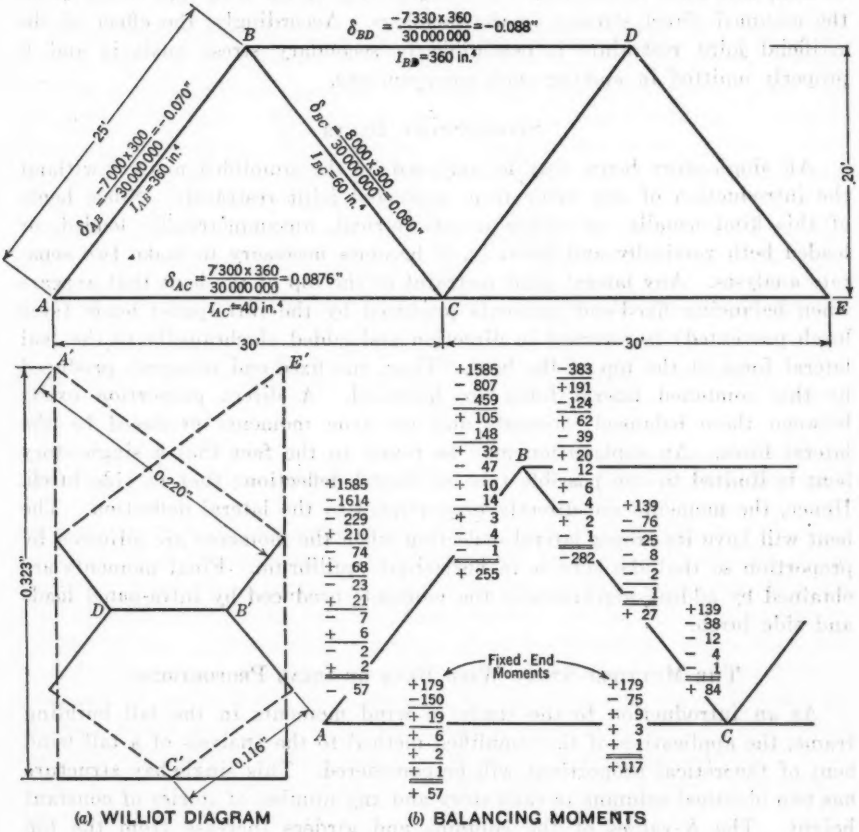
SECONDARY STRESSES IN TRUSSES

The most obvious application of the simplified method of analysis is in computing the secondary stresses in a truss. In this structure it is not necessary to guess at, or estimate, the deflections because they are caused almost entirely by the direct stresses and may be calculated directly. The computations for deflections may be made either by the Williot-Mohr diagram, or by the algebraic methods of virtual work or elastic weights. The relative deflection of the two ends of each member taken normal to its direction may be obtained from these data, and the fixed-end moments may be calculated by use of Equation (2).

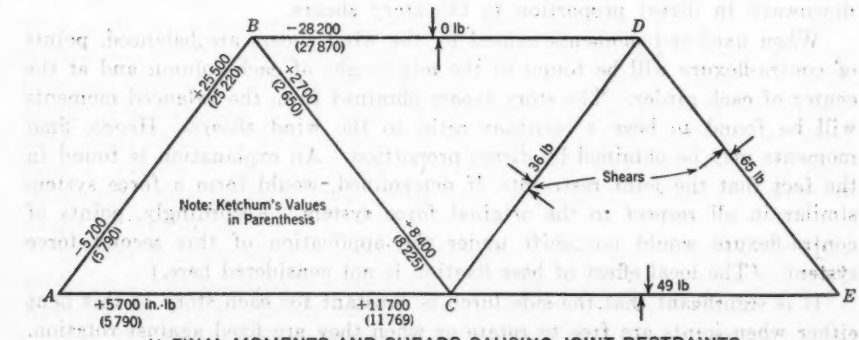
The final secondary moments are obtained by balancing moments around the joints as in any other frame. The final balanced moments are slightly in error because of the existence of small joint restraints. If one is interested in the magnitude of these joint forces, they may be calculated.

As an example of the application of the method, the secondary moments are computed in Fig. 3 for a simple truss analyzed[7] by M. S. Ketchum, M. Am. Soc. C. E. It will be seen that the moments calculated by the simplified method check Dean Ketchum's results closely. The check would naturally have been exact if inaccuracies of slide-rule and graphical computations had not entered into the work. The shears producing the joint restraints are also calculated from the secondary moments given by Dean Ketchum, and are given in Fig. 3 (*c*). The largest joint force is 156 lb., acting downward at C. If the members have reasonable areas of cross-

[7] "Steel Mill Buildings," by M. S. Ketchum, M. Am. Soc. C. E., Fourth Edition, 1921, p. 309.



(a) WILLIOT DIAGRAM (b) BALANCING MOMENTS



(c) FINAL MOMENTS AND SHEARS CAUSING JOINT RESTRAINTS

FIG. 3.—SECONDARY STRESS ANALYSIS BY SIMPLIFIED METHOD.

section, this force is considerably less than 1% of the load that would cause the assumed direct stresses in the members. Accordingly, the effect of the artificial joint restraints is negligible in secondary stress analysis and is properly omitted in making such computations.

Single-Story Bents

All single-story bents may be analyzed by the simplified method without the introduction of any error from neglected joint restraints. Since bents of this kind usually are either unsymmetrical, unsymmetrically loaded, or loaded both vertically and laterally, it becomes necessary to make two separate analyses. Any lateral joint restraint at the top of the bent that appears when balancing fixed-end moments produced by the intra-panel loads (side lurch prevented) is reversed in direction and added algebraically to the real lateral force at the top of the bent. Then, the fixed-end moments produced by this combined lateral force are balanced. A direct proportion exists between these balanced moments and the true moments produced by the lateral force. An explanation may be found in the fact that a single-story bent is limited to one possible type of lateral deflection; that is, side lurch. Hence, the moments are directly proportional to the lateral deflection. The bent will have its proper lateral deflection when the moments are adjusted by proportion so that the bent is in horizontal equilibrum. Final moments are obtained by adding algebraically the moments produced by intra-panel loads and side lurch.

The Multiple-Story Wind Bent of Ideal Proportions

As an introduction to the study of wind moments in the tall building frame, the application of the simplified method to the analysis of a tall wind bent of theoretical proportions will be considered. This single-bay structure has two identical columns in each story and any number of stories of constant height. The K-values of the columns and girders increase from the top downward in direct proportion to the story shears.

When fixed-end moments caused by the wind shears are balanced, points of contra-flexure will be found at the mid-height of each column and at the center of each girder. The story shears obtained from the balanced moments will be found to bear a constant ratio to the wind shears. Hence, final moments may be obtained by direct proportion. An explanation is found in the fact that the joint restraints, if determined, would form a force system similar in all respect to the original force system. Accordingly, points of contra-flexure would not shift under the application of this second force system. (The local effect of base fixation is not considered here.)

It is significant that the side lurch is constant for each story of this bent either when joints are free to rotate or when they are fixed against rotation. The two deflection diagrams may be represented by two straight lines of different slope. A correspondence of deflection diagrams is necessary if exact final moments are to be obtained from the first balanced moments by direct proportion.

Wind Stresses in Tall Building Frames of Regular Proportions

A tall building frame will not follow the ideal proportions of a wind bent exactly, but it is of interest that these proportions are followed closely enough so that many buildings approach a straight-line deflection curve under the action of wind loads. With this information as a background, it is possible to set up a system for analyzing wind moments in tall building frames that may be applied to any frame and that is extremely simple when applied to a frame that has a smooth deflection diagram.

First, the assumption is made that the deflections in all stories are in proportion to the story heights. This statement is identical with the mathematical relation, $\frac{\Delta}{h} = C$, in which, Δ and h represent, respectively, the increment of side deflection and the height of any story, and C is a constant. It has been shown that the fixed-end moment is determined from the relation,

$$M = \frac{6\,E\,K\,\Delta}{h} \qquad (3)$$

Therefore, if $\frac{h}{\Delta}$ is temporarily assumed to be equal to $6\,E$, the fixed-end moments in each column are equal to the K-value of that column.

These fixed-end moments are balanced in the usual manner until all joints are in equilibrium. The resulting story shears are then compared with the wind shears. If the ratios between the calculated shear and the wind shear in all stories are approximately constant—that is, if the variation does not exceed 15%—final column moments may be obtained with satisfactory accuracy by multiplying the calculated column moments in each story by the ratio of the wind shear to the calculated shear in that story. Final girder moments are obtained by dividing the sum of the column moments at a joint between the girders meeting at that joint in the same ratio as that of the uncorrected girder moments.

In certain structures one may find that the ratios of wind shear to calculated shear in adjacent stories correspond with a variation of 10 or 15%, and yet the variation between these ratios for separated stories is much greater. It is well understood that moments applied in one story of a frame have little effect upon moments in any other stories, except those adjacent to the one under consideration. Accordingly, one need compare only the ratios of wind shear to calculated shear in adjacent stories, and if these ratios do not vary from each other by more than 10 or 15%, the moments may be corrected simply by proportion.

If shear ratios in all adjacent stories change in the same direction by 15%, those for the first and sixth stories will vary from each other by 100 per cent. Nevertheless, approximately correct final moments can be obtained by proportion. The reader can prove this statement for himself in a simple manner. Consider a group of stories cut from a frame in which all columns and girders have the same K-values, all stories have equal heights, and the wind shear changes from the top downward by increments of 20% of the shear in the top story. If fixed-end moments equal to the K-values of the columns are

balanced, it will also be found that successive ratios of wind shear to calculated shear increase uniformly from the top downward by increments of 20 per cent. However, when corrected by proportion, the moments are within 11% of the true moments. It follows that a reasonable correspondence (less than 15% variation) between the ratios of actual shear to calculated shear in adjacent stories of a building frame is the only important criterion for determining when final moments may be obtained by proportion.

The foregoing discussion makes it clear that the method of balancing fixed-end moments equal to the K-values of the columns is directly applicable to practically any frame that has a reasonuably smooth deflection curve even if this curve does not approach a straight line. An interesting example is that of a set-back building. If the parts of the building above and below the set-back are reasonably regular, it is only necessary that one estimate carefully the relative deflections of the first two stories above and the first two stories below this point, in order to set the relative slopes of the two assumed straight deflection lines. (A method of estimating deflections will be developed in a later section of this paper.) From this point the analysis is unchanged except that fixed-end moments in the upper part of the building are equal to the K-values of the columns multiplied by the ratio of the slopes of the top and bottom parts of the building. The fixed-end moments in the lower section of the building are taken as equal to the K-values of the columns.

An example of the application of the simplified method in the form suggested herein (balancing fixed-end moments equal to the K-values of the columns) is given in Fig. 4. The frame selected is the lower stories of the Wilson-Maney bent that was analyzed by the slope deflection method in

TABLE 1.—Essential Dimensions and Data for Solution of Problems in Figs. 4, 5, and 8

Story	The Wilson-Maney Bent					American Insurance Union Building						
	Values of K					Values of K						
	Story height, in feet	Outside column, A	Inside column, B	Ceiling Girders: Outside bay, A-B (22 ft.)	Ceiling Girders: Center bay, B-B' (18 ft.)	Story height, in feet	Outside column, A	Inside Columns: B	Inside Columns: C	Ceiling Girders: Outside bay, A-B (14.33 ft.)	Ceiling Girders: Intermediate bay, B-C (11.12 ft.)	Ceiling Girders: Center bay, C-C' (21.33 ft.)
1....	22	25.8	25.8	30.5	37.3	22.15	42.98	39.65	59.87	135.85	191.88	293.81
2....	16	35.6	35.6	21.4	29.2	18.58	51.52	63.74	107.93	53.60	69.06	.....
3....	14	35.4	35.5	21.4	26.2	12.96	73.88	91.40	154.78	53.60	69.06	.....
4....	14	35.4	35.5	19.5	26.2	15.25	62.78	77.67	131.52	135.85	191.88	160.71
5....	14	29.4	30.4	19.5	23.8	11.82	37.84	42.91	59.30	28.81	37.12	37.14
6....	14	29.4	30.4	14.1	17.2	10.58	42.24	47.89	66.19	28.81	37.12	37.14
7....	12	28.7	30.0	12.8	15.7	11.00	40.64	46.07	63.68	28.81	37.12	37.14
8....	12	28.7	30.0	7.7	9.4	10.33	35.01	40.41	55.11	28.81	37.12	37.11

Bulletin No. 80 of the University of Illinois Engineering Experiment Station. The dimensions of this frame and the K-values of the members are given in Table 1. This structure, being of regular proportions, is well suited to analysis by the simplified method in the form described previously.

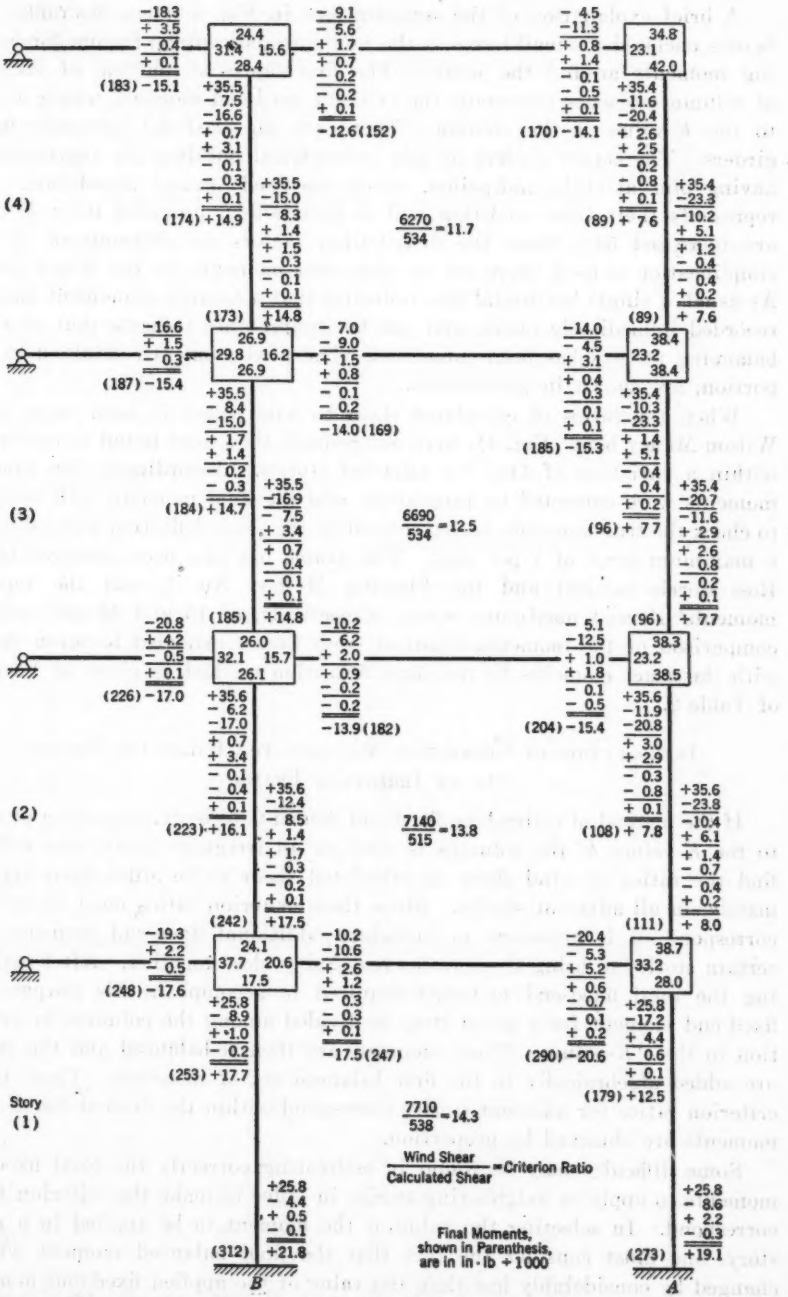


FIG. 4.—WILSON-MANEY BENT ANALYZED BY THE SIMPLIFIED METHOD BY BALANCING MOMENTS EQUAL TO THE *K*-VALUES OF THE COLUMNS.

A brief explanation of the computations in Fig. 4 seems desirable. The figures enclosed in small boxes at the joints are percentage factors for balancing moments around the joints. The first figure at the top of each row of column moments represents the original fixed-end moment, which is equal to the K-value of the column. There are no fixed-end moments in the girders. The center girders of this symmetrical building are represented as having pins at their mid-points, which are fixed against translation. This represents their true condition and is permissible, provided their K-values are increased 50% when the distribution factors are determined. If this simplification is used, there are no carry-over moments for the center girders. As usual, a single horizontal line indicates that a balancing moment has been recorded immediately above, and two horizontal lines indicate that an entire balancing process has been completed. The final moments, obtained by proportion, are shown in parentheses.

When the ratios of calculated shear to wind shear in each story of the Wilson-Maney bent (Fig. 4), were determined, they were found to correspond within a variation of 11% for adjacent stories. Accordingly, the balanced moments were corrected by proportion, and the final moments will be found to check the true moments as determined by the slope deflection method within a maximum error of 7 per cent. This frame has also been analyzed by the Ross-Morris method and the Fleming Method No. 1, and the reported moments[4] showed maximum errors, respectively, of 15 and 39 per cent. A comparison of the moments obtained from Fig. 4 extended to seven stories, with the exact moments by the slope deflection method, is given in Part (*a*) of Table 2.

Introduction of Correction Moments for Balancing Shears in an Irregular Frame

If the method of estimating fixed-end moments to be in proportion or equal to the K-values of the columns is used on an irregular frame, one will not find the ratios of wind shear to calculated shear to be alike, even approximately, in all adjacent stories. Since these criterion ratios must be made to correspond, it is necessary to introduce additional fixed-end moments into certain stories to bring these ratios reasonably close together. After estimating the total fixed-end moments required to accomplish this purpose, the fixed-end moment for a given story is divided among the columns in proportion to their K-values. These moments are then re-balanced and the results are added algebraically to the first balanced set of moments. Then, if the criterion ratios for adjacent stories correspond within the desired limits, final moments are obtained by proportion.

Some difficulty may be found in estimating correctly the total fixed-end moments to apply in neighboring stories in order to make the criterion ratios correspond. In selecting the value of the moment to be applied in a given story, one must consider the fact that the final balanced moment will be changed by considerably less than the value of the applied fixed-end moment. It is also of importance that a total positive moment applied in a given story

TABLE 2.—Comparisons of Moments as Computed by the Exact Slope Deflection Method and by the Simplified Method

(*a*) Wilson-Maney Bent* (Regular Proportions, See Fig. 4)

Story	Column *A*			Column *B*			Column *C*			Floor	Girder M_{AB}			Girder M_{BA}			Girder $M_{BB'}$		
	Slope deflection	Simplified method	Percentage, error	Slope deflection	Simplified method	Percentage, error	Slope deflection	Simplified method	Percentage, error		Slope deflection	Simplified method	Percentage, error	Slope deflection	Simplified method	Percentage, error	Slope deflection	Simplified method	Percentage, error
7																			
	60.0	56	7	124.0	123	1	.....	....	....	6	135.0	131	3	125.6	123	2	141.5	141	0
	75.0	75	0	143.3	141	2	.....	....	....										
6																			
	83.2	84	1	155.7	156	0	.....	....	....	5	167.7	170	1	152.0	154	1	166.0	166	0
	84.5	86	2	162.3	164	1	.....	....	....										
5																			
	82.8	81	2	162.0	161	0	.....	....	....	4	170.5	170	0	153.6	152	1	184.4	183	1
	87.8	89	1	176.1	174	1	.....	....	....										
4																			
	87.8	89	1	173.5	173	0	.....	....	....	3	187.1	185	1	171.7	169	2	189.6	187	1
	100.3	96	4	187.5	184	2	.....	....	....										
3																			
	90.7	96	6	182.0	185	2	.....	....	....	2	203.0	204	0	184.0	182	1	226.0	226	0
	113.0	108	4	226.5	223	2	.....	....	....										
2																			
	107.5	111	3	239.0	242	1	.....	....	....	1	287.8	290	1	245.0	247	1	247.5	248	0
	178.8	179	0	251.0	253	1	.....	....	....										
1																			
	272.0	273	0	308.2	312	1	.....	....	....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....

(*b*) American Insurance Union Building† (With Correction Moments for Balancing Shears, See Fig. 5)

											Girder *AB*			Girder *BC*			Girder *CC'*		
5	183	18	2	382	39	2	439	46	5	6	365	38	4	400	41	3	432	47	9
	196	18	8	389	38	2	452	43	5		332	34	2	416	43	3			
	184	18	2	378	38	1	437	44	1		380	36	5	421	42	0			
4										5							452	44	3
	186	17	9	383	38	1	445	44	1		347	34	2	436	43	1			
	188	20	6	360	37	3	421	43	2		374	37	1	406	41	1			
3										4							441	44	0
	294	29	1	471	46	2	571	55	4		338	34	1	424	43	1			
	362	36	1	678	67	1	825	81	2		655	65	1	631	62	2			
2										3							677	66	3
	239	25	5	526	53	1	427	43	1		518	51	2	719	70	3			
	333	31	7	702	68	3	391	37	5		572	56	2	707	71	0			
M										2							.....	....	....
	301	30	0	681	69	1	269	30	12		520	50	4	817	80	2			
	332	34	2	678	66	3	670	64	4		632	64	1	792	79	0			
1										*M*							.....	....	....
	400	40	0	802	80	0	1 078	109	1		567	56	1	938	94	0			
	583	57	2	682	67	2	1 000	98	2		983	97	1	745	74	1			
0										1							1 285	128	0
	713	71	0	730	72	1	1 086	108	1		738	73	1	791	79	0			

(*c*) American Insurance Union Building† (Deflections Estimated by Use of *n*-Values, See Table 3 and Fig. 8)

											Girder *A*			Girder *B*			Girder *C*		
5	183	20	9	382	39	2	439	44	0	6	365	36	1	400	38	5	432	42	3
	196	20	2	389	36	7	452	43	5		332	32	4	416	41	1			
	184	19	3	378	37	2	437	44	1		380	39	3	421	40	5			
4										5							452	45	0
	186	20	8	383	37	3	445	44	1		347	33	5	436	42	4			
	188	19	1	360	36	0	421	42	0		374	39	4	406	40	1			
3										4							441	44	0
	294	29	1	471	46	2	571	58	2		338	34	1	424	42	1			
	362	36	1	678	66	3	825	82	1		655	65	1	631	62	2			
2										3							677	68	0
	239	24	0	526	52	1	427	44	3		518	50	3	719	72	0			
	333	33	1	702	71	1	391	38	3		572	57	0	707	71	0			
M										2							.....	....	....
	301	30	0	681	69	1	269	26	3		520	52	0	817	82	0			
	332	32	4	678	68	0	670	68	1		632	63	0	792	81	2			
1										*M*							.....	....	....
	400	40	0	802	79	1	1 078	107	1		567	56	1	938	94	0			
	583	56	4	682	68	0	1 000	99	1		983	96	2	745	74	1			
0										1							1 285	126	2
	713	70	2	730	73	0	1 086	108	1		738	73	1	791	80	1			

* Slope deflection moments were computed by Wilson and Maney and are given in inch-pounds ÷ 1 000.
† Slope deflection moments were computed by Ross and Morris and are given in inch-pounds ÷ 10 000.

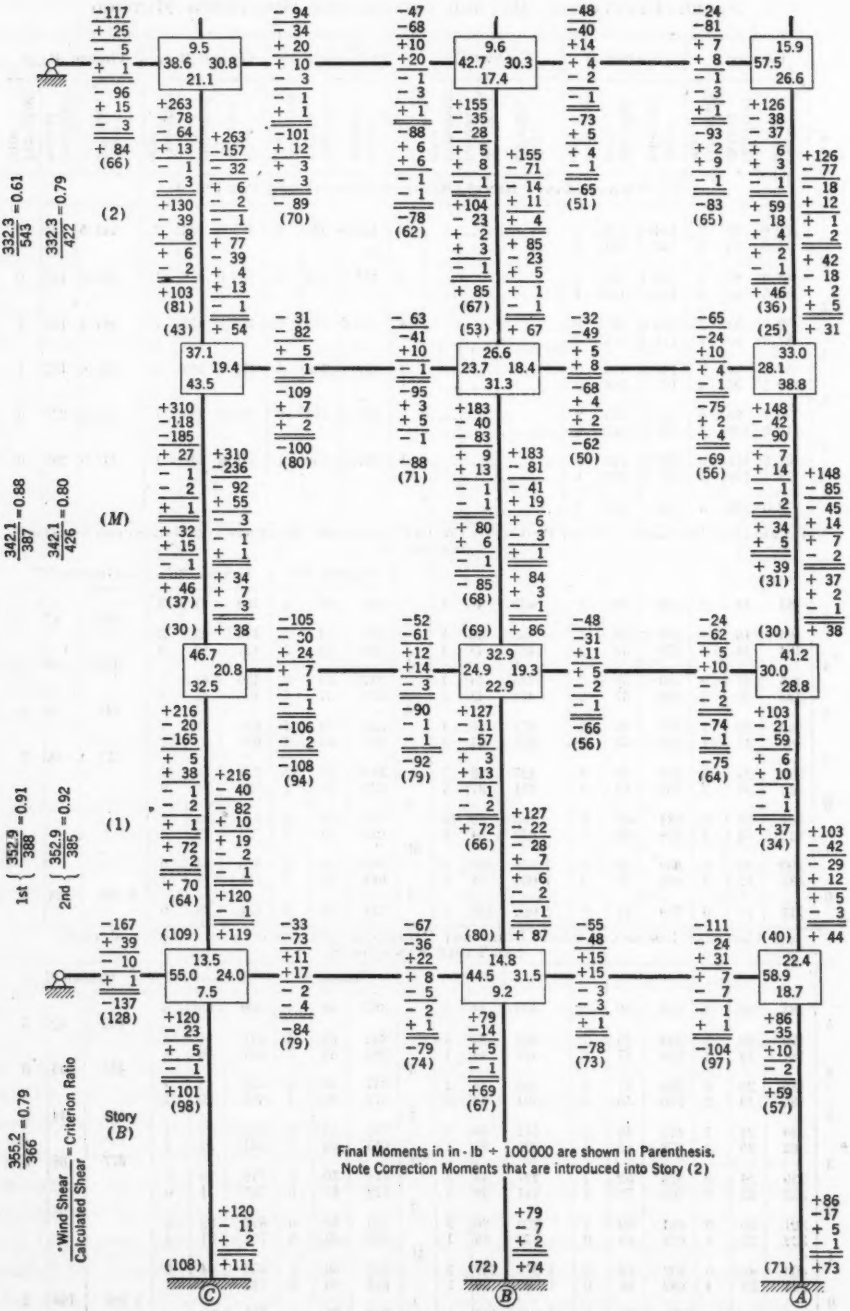
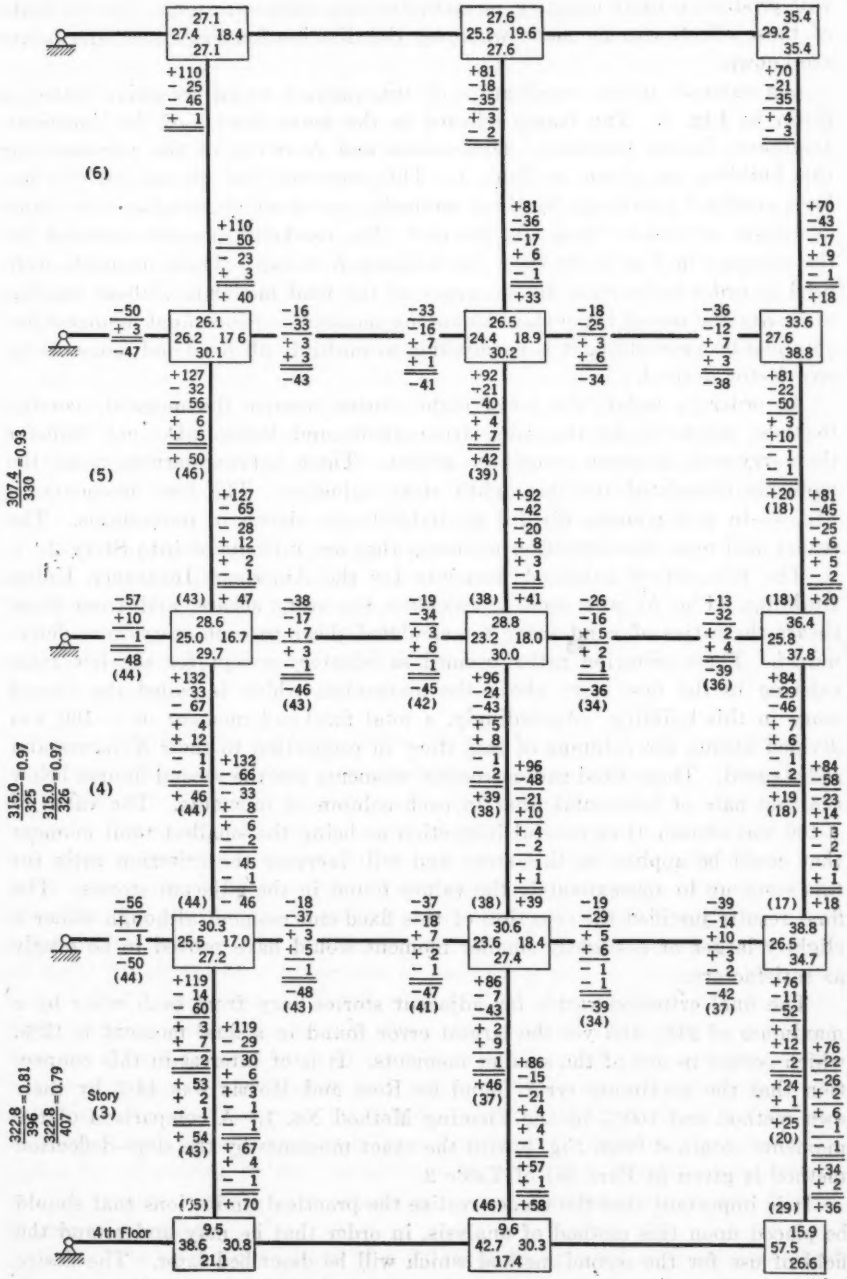

Final Moments in in-lb ÷ 100 000 are shown in Parenthesis.
Note Correction Moments that are introduced into Story (2)
Story (B)
Wind Shear / Calculated Shear = Criterion Ratio
(C)
(B)
(A)

FIG. 5.—AMERICAN INSURANCE UNION BUILDING ANALYZED BY THE SIMPLIFIED METHOD MOMENTS APPLIED



OF BALANCING MOMENTS EQUAL TO TWICE THE *K*-VALUES OF THE COLUMNS IN STORY 2.

will produce a total negative moment in each adjacent story. An estimate of these effects can be made wherever the distribution factors at the joints are known.

An example of the application of this method to an irregular frame is given in Fig. 5. The frame selected is the lower stories of the American Insurance Union Building. Dimensions and K-values of the members for this building are given in Table 1. This structure was chosen since it has been analyzed previously by three methods, one of which was the exact slope deflection method by Ross and Morris.[4] The fixed-end moments selected for the columns in Fig. 5 are twice the Column K-values. These moments were used in order to increase the accuracy of the final moments without making necessary the use of three-place balancing moments. Since final moments are obtained by proportion, it is permissible to multiply all fixed-end moments by any factor desired.

In order to isolate the lower eight stories, assume the moments coming to these joints to be the same from above and below. Do not transfer the carry-over moment across the girder. These approximations make the moments calculated for the eighth story valueless. The final moments in Fig. 5—in inch-pounds, divided by 100 000—are shown in parenthesis. The reader will note the correction moments that are introduced into Story 2.

The first set of balanced moments for the American Insurance Union Building, (Fig. 5) was used to calculate the story shears, and from these shears the ratios of wind shear to calculated shear in each story were determined. These criterion ratios seemed satisfactory except for the low ratio existing in the first story above the mezzanine, which is called the second story in this building. Accordingly, a total fixed-end moment of —160 was divided among the columns of this story in proportion to their K-values and re-balanced. These fixed-end correction moments are the second figures below the first pair of horizontal lines in each column of moments. The value of —160 was chosen after careful inspection as being the smallest total moment that could be applied in this story and still increase the criterion ratio for this story up to approximately the values found in the adjacent stories. The final results justified the selection of this fixed-end moment although either a slightly larger or a slightly smaller moment would have proved to be nearly as satisfactory.

The final criterion ratios for adjacent stories vary from each other by a maximum of 21% and yet the largest error found in a final moment is 12%, which occurs in one of the smaller moments. It is of interest in this connection that the maximum error found by Ross and Morris was 48% by their own method and 100% by the Fleming Method No. 1. A comparison of the moments obtained from Fig. 5 with the exact moments by the slope-deflection method is given in Part (*b*) of Table 2.

It is important that the reader realize the practical limitations that should be placed upon this method of analysis, in order that he may understand the field of use for the second method which will be described later. The desire of the writer has been to reduce wind stress analysis to the simple process of balancing one set of fixed-end moments and to eliminate the need for balanc-

ing shears with the corresponding introduction of a second, third, and fourth set of fixed-end moments which must be balanced in their turn. The foregoing method of analysis is ideal, wherever the frame deflects into a reasonably smooth curve. For such a structure, the analysis is completed by balancing a single set of fixed-end moments and then by correcting the balanced moments by proportion. Even if there is a considerable irregularity in one or two stories, the method is still advantageous since proper correction moments can be introduced into these stories, re-balanced with small effort, and the moments corrected by proportion as for a regular frame. However, if more than two neighboring stories of the frame are highly irregular, it becomes rather difficult to estimate the proper fixed-end correction moments. In fact, one may err to such an extent in selecting these moments that, after re-balancing, the shear ratios for adjacent stories still do not correspond within the desired range of variation, and it becomes necessary to introduce a second set of correction moments. Naturally, such mistakes are most likely to occur in an analysis performed by one who is unfamiliar with the method.

After making use of this method of balancing fixed-end moments in proportion to the K-values of the columns, the reader will find that its field of major importance is in the analysis of those frames for which an inspection of the arrangement of the members and of their K-values indicates that correction moments probably will be required in one or two stories only. If this inspection indicates that correction moments will be required in a large number of stories, it seems desirable to make a closer estimate of the true relative deflections, in order that the introduction of correction moments may be avoided.

Wind Stresses in Tall Building Frames of Irregular Proportions

In the method of analysis just presented, the story deflections of the frame are assumed to be in proportion to the story heights. The analysis of an irregular frame by this system is complicated subsequently by the need for introducing correction moments into one or more stories of the frame. A method of estimating the true story deflections would seem to offer an advantage since the need for the introduction of correction moments might be eliminated in this manner. In fact, final moments can be obtained within an error of about 6% if the side deflections of each story can be estimated within a variation of 10 per cent. The following method of computing deflections has been found to give approximately this accuracy in all except highly irregular frames. As a maximum error of even 10 or 12% in the wind moments would not be objectionable ordinarily, this method may be used in practical design even for extremely irregular structures. Clearly, this system of estimating deflections may be combined with the conception of producing equal criterion ratios for all stories, by the introduction of correction moments into certain stories, to give a method for determining wind moments within any desired degree of precision.

Since the lateral deflections of a building under wind load are dependent upon the properties of all the columns and all the girders, it seems reasonable that the deflections might be estimated with considerable accuracy by

considering the deflections of a single combined column restrained by single girders at each floor level. The K-value of the combined column for each story is taken as the summation of the K-values of all columns in that story, or ΣK_c. The combined girder at each floor level has a K-value equal to twice the sum of the K-values of all girders at that floor, or $2\Sigma K_g$. The factor, 2, is required because each girder is effective in resisting the moments at two joints. The problem is now reduced to estimating the lateral deflections of this single column with single restraining girders at each floor level.

It is necessary, first, to consider the relation between moment, deflection, and joint rotation in a single story. The lateral deflection of the combined

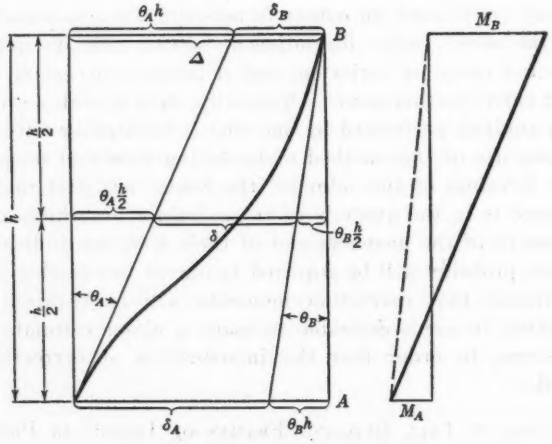


FIG. 6.—COLUMN DEFLECTION.

column may be looked upon as made up of two parts: (1) The angle change at the far end times the story height; and (2) the deflection at the near end from a tangent drawn at the far end. However, considering the deflection as divided at the mid-height, as shown in Fig. 6, it becomes clear that,

$$\Delta = \frac{\theta_A + \theta_B}{2} h + \frac{\delta_A + \delta_B}{2} = \theta_{avg} h + \delta$$

but δ is the average of the deflections at the two ends measured from tangents drawn at the far ends. Hence, δ is readily computed by area moments; thus,

$$\delta = \frac{1}{2EI}\left(\frac{M_A h^2}{6} + \frac{M_B h^2}{6}\right) = \frac{(M_A + M_B) h}{12 E K}$$

The sum of M_A and M_B for the combined column is equal to the total moment, M_t, in the story, which is equal to the shear times the story height, and the K-value of the combined column is ΣK_c. Hence, $\delta = \dfrac{M_t h}{12 E \Sigma K_c}$, and,

$$\Delta = \theta^{avg} h + \frac{M_t h}{12 E \Sigma K_c} \quad \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

Equation (4) can also be derived from the slope-deflection equations.

The second term of the expression for Δ is unaffected by the division of M_t into M_A and M_B. However, the first term, which involves the individual values of θ, is dependent upon the division of the total moment between the top and the bottom of the column. When the structure is quite regular, it is sufficiently accurate to assume that $M_A = M_B = \frac{M_t}{2}$. In an analysis of the Wilson and Maney bent this assumption was made, and the computed wind moments checked the true moments within a maximum error of 2 per cent. Nevertheless, since the use of fixed-end moments equal to the K-values of the columns is sufficiently accurate for a frame of reasonable regularity, a better approximation of the true division of the total moment between the two ends of the combined column seems desirable. At least, the refinement suggested subsequently will prove advantageous when the stiffness ratios between girders and columns at the two ends of a combined column vary from each other by more than 50 per cent.

The "stiffness ratio of girders to columns" can be defined as:

$$r = \frac{2\,\Sigma K_g \text{ of the girders between Stories } A \text{ and } B}{\Sigma K_c \text{ of the columns in Story } A \text{ plus } \Sigma K_c \text{ in Story } B}$$

This ratio will be called the r-ratio for any floor level.

Let $n_{top} = 2\,r_{top} + r_{bottom}$ and $n_{bottom} = 2\,r_{bottom} + r_{top}$. By checking the moments in several building frames, it has been found that the total column moments at the top and bottom of a story are to each other approximately as the n-values at the top and the bottom of the story. Accordingly, the total column moment is divided as follows:

$$M_{top} = M_t \frac{n_{top}}{n_{top} + n_{bottom}}, \text{ and } M_{bottom} = M_t \frac{n_{bottom}}{n_{top} + n_{bottom}}$$

Note that the n-value at the top of a basement column is ∞ and that the n-value at the bottom is $2\,\infty$. Hence, the total column moment would be divided in the ratio of 67% to the bottom and 33% to the top. This naturally assumes a fixed base at the bottom of the basement column. Economy usually requires that the moments at the top and the bottom of the basement columns correspond more closely than 2 to 1, and, accordingly, it seems desirable to place an upper limit of 60% on the division of moment.

By adding the total column moments above and below any floor level, the moment in the girder, M_g, is obtained. Since the K-value of the combined

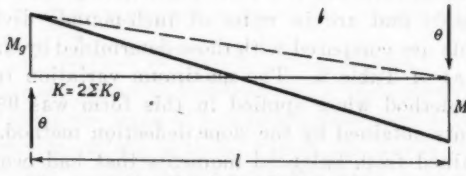


FIG. 7.—JOINT ROTATION.

girder is known, its angle change or the joint rotation can be computed by area moments if the assumption is made that there is a point of contra-flexure

at its mid-point. For this case, from Fig. 7,

$$\theta = \frac{M_g l}{6\,EI} = \frac{M_g}{6\,E\,2\Sigma K_g} = \frac{M_g}{12\,E\Sigma K_g} \quad \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

When all the θ-values have been determined, it is possible to compute the values of Δ for each story from the known relation,

$$\Delta = \theta_{avg}\, h + \frac{M_t h}{12\,E\,\Sigma K_c} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

The fixed-end moments in the individual columns caused by the side lurch are calculated from Equation (3), in which, K is successively the stiffness ratio for each individual column. There are no fixed-end moments in the girders. The fixed-end moments in the columns are balanced in the usual manner around the joints to obtain the first set of balanced moments.

When the balanced moments have been obtained, the column shears are calculated and added together. The total shear should equal the wind shear, and the difference is a measure of the error in the uncorrected moments. If these shears are within 10% of the true shears, the final moments can certainly be obtained with sufficient accuracy by proportion (maximum errors of about 6%). The final column moments are proportioned so as to make the calculated shears check the wind shears. The final girder moments are obtained by dividing the sum of the final column moments at a joint between the girders in the same ratio as that of the uncorrected girder moments.

If the first computed shears in certain stories vary from the true wind shears by more than 10 or 15%, the designer must use his judgment as to whether the foregoing method of correcting the moments by proportion will be sufficiently accurate to meet his requirements or whether it will be necessary to introduce correction moments into certain stories, as has been described in a preceding section. It seems improbable that the important moments obtained by proportion would ever be in error more than 10 or 12% when the calculated shears checked the true shears within 20 per cent. If continued experience with the method justifies this assumption, the need for introducing a set of correction moments probably will never arise.

The lower eight stories of the American Insurance Union Building were analyzed by the simplified method making use of the n-values as explained. The computations of the fixed-end moments for the lower eight stories are given in Table 3. The fixed-end moments are balanced, criterion ratios are computed, and final moments are determined in Fig. 8. Final moments are shown in parenthesis and are in units of inch-pounds divided by 100 000. These final moments are compared with those determined by the slope deflection method in Part (*c*) of Table 2. The maximum variation in moment found by the simplified method when applied in this form was 9% when checked against the moments obtained by the slope deflection method. Note that this accuracy was obtained from balanced moments that had been carried out to two places only. It will be recalled that this same building was analyzed by the first form of the simplified method, in which fixed-end moments in proportion to the K-values of the columns are balanced, and that the maximum error in the final moments was 12 per cent. It will also be recalled that

TABLE 3.—COMPUTATIONS FOR ESTIMATING STORY DEFLECTIONS, AMERICAN INSURANCE UNION BUILDING, ANALYSIS COMPLETED IN FIG. 8

Story	Height, in feet	ΣK_c, in inches³	ΣK_g, in inches³	r	n	Moment division, percentage	Shear, in kips	Total moment, M_t, in kip-feet	Total column moments, in kip-feet	$E\theta \div 1\,000$	Deflections, in inches: $E\theta_{avg.} k \div 1\,000$	$E\delta \div 1\,000$	$E\Delta \div 1\,000$	Fixed-end column moments, in inch-pounds ÷ 10 000: A	B	C
			168.9	0.65						18.3						
6	10.33	261.0			1.90	51	300.0	3 100	1 580		2 330	1 470	3 800	643	743	1 012
					1.85	49			1 520							
			168.9	0.60						19.3						
5	11.00	300.8			1.75	51	307.4	3 380	1 730		2 570	1 480	4 050	746	848	1 173
					1.70	49			1 650							
			168.9	0.55						19.6						
4	10.58	312.6			1.67	50	315.0	3 330	1 660		2 460	1 350	3 810	760	864	1 193
					1.69	50			1 670							
			168.9	0.57						19.2						
3	11.82	280.0			3.12	41	322.8	3 820	1 570		1 820	1 940	3 760	600	681	942
					4.53	59			2 250							
			816.1	1.98						6.5						
2	15.25	544.0			4.37	60	332.3	5 070	3 040		2 160	1 710	3 870	796	986	1 670
					2.80	40			2 030							
			245.4	0.41						17.1						
M	12.96	640.2			1.27	49	342.1	4 430	2 170		2 880	1 070	3 950	1 128	1 393	2 360
					1.31	51			2 260							
			245.4	0.45						19.9						
1	18.58	446.2			3.50	40	352.9	6 560	2 620		3 050	3 270	6 320	876	1 083	1 838
					5.66	60			3 940							
			949.2	2.60						7.5						
B	22.15	285.0			∞	40	355.2	7 870	3 150		990	7 340	8 330	809	745	1 130
			∞	∞	2_∞	60			4 720	0						

correction moments were introduced into one story in this analysis. Accordingly, the second form of the simplified method, which involves a preliminary estimate of story deflections, was more satisfactory for the analysis of this structure and will usually be found advantageous for the analysis of such irregular frames.

The time consumed by the two variations of the simplified method was not very different for this frame. In the first analysis, time was required for the introduction of correction moments into the frame, while in the second analysis about an equal amount of time was required to make the preliminary estimate of deflections. In either case, there is an appreciable saving over the time consumed by the general method of successive corrections, although the saving is not as important as was found for the more regular frame.

EFFECT OF COLUMN DEFORMATION

Since the windward columns lengthen while the leeward columns shorten under wind load, there are fixed-end moments produced in the girders by this effect. The obvious procedure is to determine the direct stress deformations in the columns after having determined the wind moments. From these deformations the fixed-end girder moments are computed and balanced. These balanced moments must be made a part of the original analysis, and since they produce story shears, the criterion ratios will be changed. The final

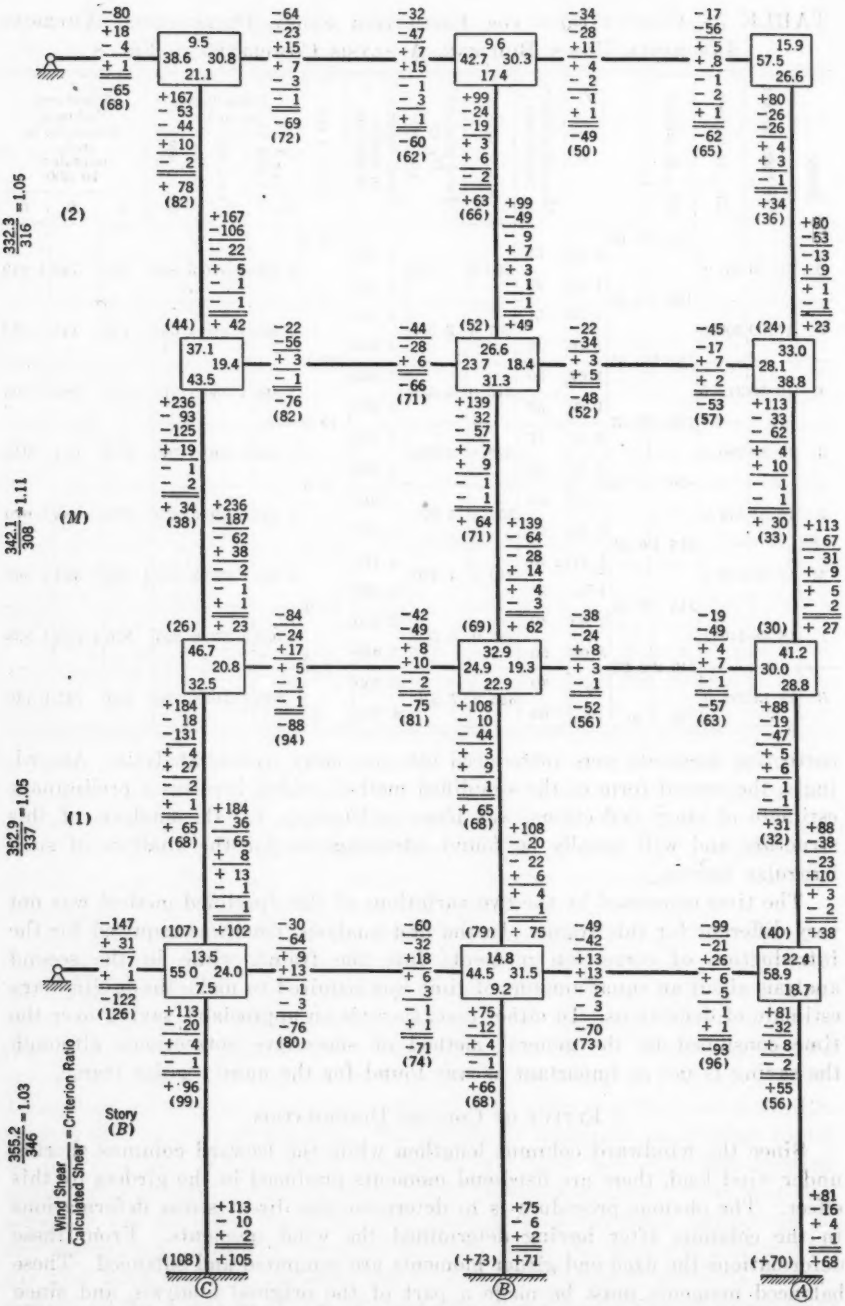


FIG. 8.—BALANCING FIXED-END MOMENTS OBTAINED FROM THE DEFLECTIONS ESTIMATED IN TABLE 3, AMERICAN INSURANCE UNION BUILDING.

moments are these balanced moments plus the wind moments obtained by proportion using the new criterion ratios.

Naturally the direct stresses, if re-computed, would be found slightly changed, but a further revision of moments would seem to be unwarranted. In fact, it is only in the tower structure that column deformation becomes of importance in wind stress calculations. The writer has estimated the possible effect of column deformation in a discussion[8] of the Second Progress Report of Sub-Committee No. 31, Committee on Steel of the Structural Division, on Wind-Bracing.

The effect of column deformation can be included directly in a second analysis whenever a revision of the sizes of the members makes a second analysis necessary. The approximate direct stresses in the columns are then known, and the fixed-end girder moments may be introduced and balanced along with the fixed-end column moments. Note, however, that the column moments must have been obtained from absolute rather than relative story deflections. Otherwise, the effect of column deformation would be either exaggerated or minimized when final moments are obtained by proportion.

The Open-Web or Vierendeel Truss

This structure is rather unusual, but does occur where clear openings are required in building design and has been used as a bridge structure, largely in Europe. Its analysis for fixed loads may be made by the simplified method either in the form of balancing fixed-end moments in proportion to the K-values of the members or by balancing fixed-end moments obtained from the estimated panel deflections.

The analysis of the open-web truss by balancing fixed-end moments equal to the K-values of the members is almost identical to the similar process for a building frame. In this structure the fixed-end moments occur only in the horizontal members or chords, provided that the effect of direct stress deformation is to be neglected, as is usual. Depending upon the number of panels in the truss, it may or may not be necessary to introduce correction moments into the various panels before adjusting moments by proportion. The criterion ratios for adjacent panels must be made to correspond within about twice the maximum variation to be expected between the calculated moments and the true moments. In other words, if a 5 or 6% error is permissible in the final moments, a 10% variation may be allowed between the criterion ratios for adjacent panels.

It will be an obvious improvement to assume that a parabolic or circular deflection curve exists rather than one of triangular shape. The latter is assumed automatically to be the deflection curve if all fixed-end moments are taken as equal to the K-values of the chords. A parabola will serve as well as any other standard curve, and its properties are most readily determined. From the relative assumed deflections of the panel points, one determines the ratios for the various panels by which the K-values of the chords must be multiplied to obtain the original fixed-end moments. The process is simple enough to make a detailed explanation seem unnecessary.

[8] *Proceedings*, Am. Soc. C. E., May, 1932, p. 956.

Use of the Simplified Method of Wind Stress Analysis in the Design of a Tall Building Frame

The simplified method of wind stress analysis is convenient for use in the design of a tall building frame. Any statically determinate method of analysis with which the designer is familiar may be used in estimating the wind moments and direct stresses to be used in a preliminary design of the structure. The statically determinate method of analysis must be supplemented by the designer's judgment wherever there are extreme irregularities in the frame. When the sizes of the members have been determined for the most serious combination of dead load, live load, and wind, the simplified method of analysis should be used to determine the true wind moments in this frame.

Slight revisions of the proportions of the frame will probably be necessary to care for the new distribution of wind moments. If the preliminary design was made by an experienced designer, these changes will usually be so unimportant that a new analysis will not be required. Where considerable change is found necessary in the sizes of a few members, it may be desirable to re-distribute the moments about the joints into which these members frame. If radical changes must be made in the sizes of a number of members, it will be necessary to repeat the analysis. However, in this second analysis, the designer has at his command an almost exact knowledge of the story deflections (obtainable from the data of the first analysis), and, therefore, the labor of the second analysis is slightly reduced.

This outline of design procedure makes it perfectly clear that the experienced designer has a real advantage over the beginner in the use of this method. The design of any indeterminate structure should be simplified by a previous knowledge of correct proportions. Any method of design in which preliminary proportions are set essentially by rule of thumb does not take full advantage of the reduction of effort that experience should offer.

Naturally, the question of economy enters into the problem of establishing the proportions of the structure. Minimum wind moments will occur when points of contra-flexure exist at the centers of the members, and the smallest total direct stresses in the columns occur when the direct stresses in the interior columns are zero. However, a wind stress of from 25 to 50% of the dead load and live load stress in a member is usually allowed without requiring an increase in the size of the member. Accordingly, it is not necessarily true that there will be an important economy obtained by forcing the points of contra-flexure to the centers of the members or by proportioning the structure so that there will be zero direct stresses in the interior columns. In fact, the opposite may readily be true.

Near a point of extreme irregularity in the structure, the designer should study the matter of proportions carefully. For instance, when the lower story columns are fixed at their bases, it is frequently considered desirable to use heavy girders at the level of the first floor to force the points of contra-flexure in the lower story columns down near their mid-height. The importance of this arrangement increases with increase in the height of

the building. However, one should realize that a heavy girder at the first floor will also require a reasonably heavy girder at the second floor, neither of which can be stressed up to its allowable stress. This heavy second floor girder is required in order to keep the points of contra-flexure in the second-story columns near their mid-height. The question arises as to whether or not it is more economical to place excess material in these first and second-floor girders rather than to increase the sizes of the first-story columns to care for heavy wind moments. This example is typical of the kind of problem that the designer must investigate around each point of extreme irregularity in the frame. A knowledge of the simplified method is extremely valuable in this connection.

Conclusion

This paper has been limited to a presentation of a general system of analysis for stresses in continuous frames that undergo joint translations but in which the loading is fixed. An analysis for the effect of moving loads is an entirely different matter and is not considered. The reader will probably see such an application in the calculation of the maximum stresses for a Vierendeel bridge truss. Since the value of this method of analysis is dependent upon its usefulness to the designer, the writer has indicated briefly its application to the design of the most important structure in this field—the tall building frame. A discussion of the proper proportions of building frames was intentionally omitted from the paper, although a study of the relative economy of frames having various deflection curves would be both interesting and valuable.

Since certain examples mentioned in this paper have been solved to within a range of error of 2 or 3%, it is feared that the reader may draw the conclusion that this precision is put forward as desirable. Such precision is clearly wasted because even the physical properties of materials are not known to this degree of exactness. On the other hand, it seems to the writer that the entire attitude of the designer must be demoralized by the knowledge that his tool for wind-stress analysis may be in error by from 50 to 100 per cent. Careless design would be the natural result. Any tool for wind stress analysis, such as the simplified method, by which the designer can compute the wind stresses to conform to a predetermined standard of accuracy, should tend to increase his confidence in the value of his calculations and improve the quality of his design work. Compared with the importance of this result, the value of a discussion as to whether the standard of accuracy in wind stress analysis should be set at a maximum error of 5, 10, or 20% seems trivial.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS

GEORGE WASHINGTON BRIDGE: CONSTRUCTION OF SUBSTRUCTURE

BY MONTGOMERY B. CASE,[1] M. AM. SOC. C. E.

SYNOPSIS

The principal features of the construction methods used on the foundations for the main piers and the anchorages for the cables of the George Washington Bridge are described in this paper. The foundation piers for the New Jersey tower involved the use of open coffer-dams, larger and deeper than any previously used on bridge construction. In New Jersey, the anchorage for the cables was obtained by excavating inclined tunnels in the solid rock of the Palisades in which the anchorage steel was embedded in concrete. The existence of Jeffreys Hook projecting into the Hudson River in the central part of Fort Washington Park provided an ideal site for the location of the piers for the tower, and the natural topography behind the point and adjacent to Riverside Drive offered adequate footing for the massive concrete anchorage at the New York end of the main bridge.

Preliminary studies had brought out clearly the many advantages of the location passing over the extreme point of Fort Washington Park. The narrowing of the river at this point made possible a shorter main span than at any other location in this vicinity. The ground on both sides of the river is high. On the New York side the sound rock forming Fort Washington Point offered excellent foundations for the tower, and the frequent outcroppings and exposed ledges at the site of the anchorage indicated that a rock foundation could be obtained under the entire area of the anchorage with relatively little excavation.

NEW JERSEY TOWER FOUNDATIONS

The preliminary borings for the foundation of the New Jersey tower had indicated that the piers could be founded on rock at a depth not to exceed

NOTE.—Discussion on this paper will be closed in April, 1933, *Proceedings*.

[1] Engr. of Constr., The Port of New York Authority, New York, N. Y.

100 ft. if they could be located near the shore line. While the pierhead line established by the War Department would have permitted the location of this bridge pier about 150 ft. farther into the stream, the studies indicated clearly that the difficulties and increased cost of carrying this pier to rock at the greater depth would more than offset any saving in the length of the main span.

The area of the base of the tower is so large that from the first it was apparent that, with the firm rock foundations available, it was not necessary to build a single large pier. The middle third could be omitted so that the foundation could take the form of a pier under each of the two legs of the tower.

The size and shape of the foundations were thus controlled to a considerable extent by the design of the tower. The adopted design provided a block of concrete under each tower leg as shown in Fig. 1. It is to be noted that the piers are faced with granite from the top to a point 7 ft. below mean low water.

The vertical load on the foundation amounts to an average of 22 tons per sq. ft. Under the most unfavorable combination of vertical and longitudinal forces the resultant will have an eccentricity of 5.2 ft., or only 5% of the width of the tower foundation. This will cause a maximum edge pressure on the rock foundations of 28 tons per sq. ft., or about 400 lb. per sq. in. without considering buoyancy.

Compression tests made of various pieces of cores from different rock strata indicate a variation in the strength of the rock of between 3 000 and 24 000 lb. per sq. in. and a probable average of the entire mass underlying the tower foundation of between 12 000 and 15 000 lb. per sq. in. The lowest test result obtained is, therefore, nearly eight times the maximum edge pressure and ten times the average pressure. The average strength of the rock is probably more than thirty times the edge pressure that will be transmitted by the mass concrete.

The early studies based on the preliminary borings contemplated that the pier would be sunk by the pneumatic process which, with the information then available, appeared to be the safest and most practicable method. However, the final borings indicated that the maximum depth to rock was only 75 ft. and the average depth less than 50 ft., which made it desirable to give consideration to the use of an open coffer-dam.

On account of the slope of the rock, if the pneumatic process had been used, it would have been desirable to divide the area of each pier into two or more caissons to avoid the necessity of excavating perfectly sound rock along the high side next to the shore line in order to carry the cutting edge within reach of the rock along the outboard or river edge of the caisson. The difficulty of obtaining effective connections between these caissons to insure a unified action under a single pier was recognized. This difficulty would not have to be met if the open coffer-dam was used, which method would allow a careful and reliable preparation of the rock bed upon which the foundation was to be placed.

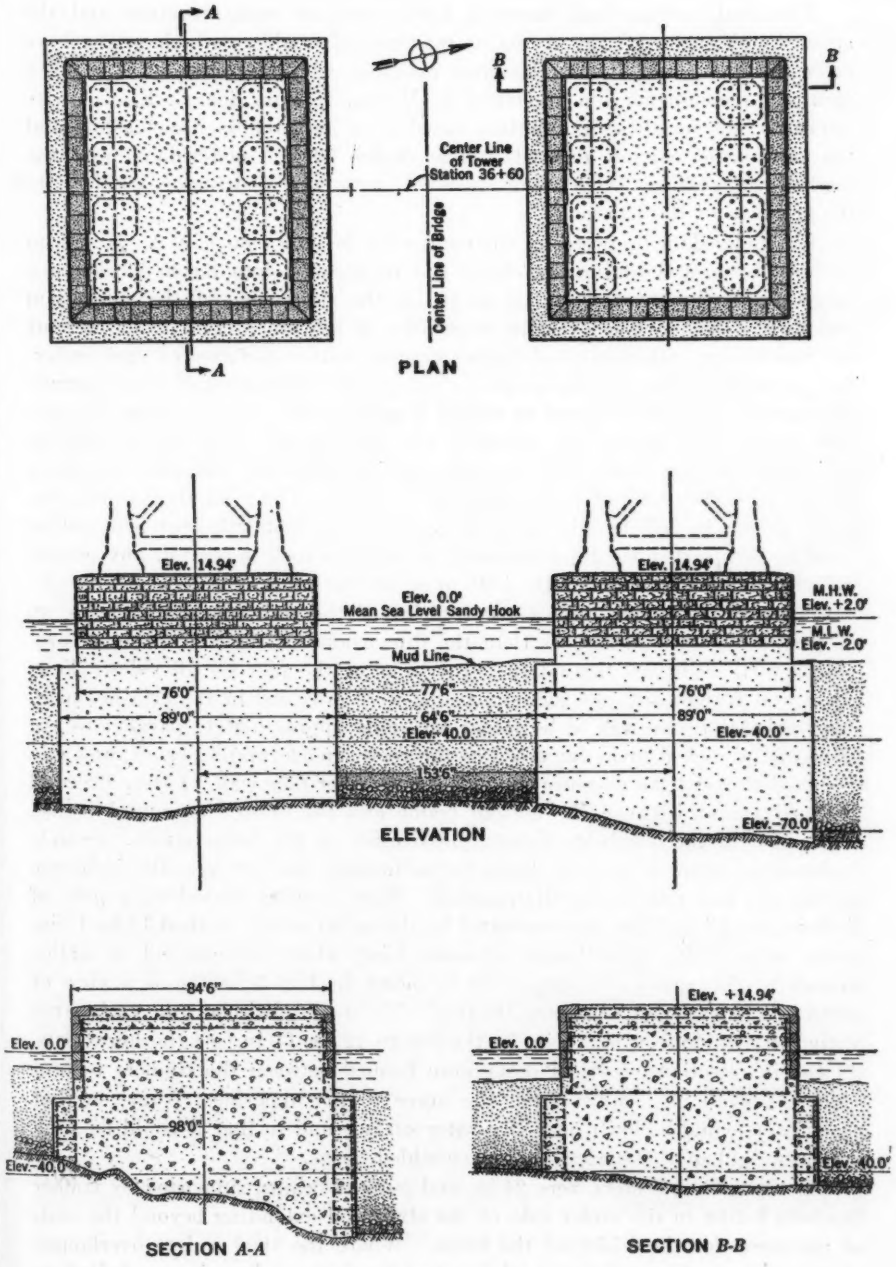

FIG. 1.—SECTIONS SHOWING DESIGN OF NEW JERSEY TOWER FOUNDATIONS.

The final borings had shown a fairly compact rock structure and the apparent absence of large seams or evidence of boulders which might have made the open coffer-dam method very difficult. Consequently, it was decided, upon the recommendation of Daniel E. Moran, M. Am. Soc. C. E., to invite alternate bids for these foundations based upon both the open coffer-dam and the pneumatic caisson methods. The design of the coffer-dams and the method of sinking them were optional with the contractor, subject to the approval of the Engineer.

The coffer-dam proposed by the contractor is shown on Fig. 2. It was to be built by first dredging the entire site of the two foundation blocks as a single excavation to a depth of 45 ft., at the river edge of the coffer-dam and to the rock surface over the remainder of the area. This would permit the assembling and sinking of timber bracing within the area of each coffer-dam previous to the driving of steel sheet-piling. This method would permit the bracing to be built down to within a satisfactory distance from the surface of the rock before any pumping was started and would avoid creating the condition in which the sheeting has considerable pressure to resist before the bottom sets of braces have been placed. The steel sheet-piling was to be driven in full lengths with the expectation that sufficient penetration could be obtained in the top layers of disintegrated rock to prevent any serious difficulty from inflow under the bottom of the sheeting.

The over-all dimensions of the interior of the coffer-dam were made 10 ft. greater in both directions than the dimensions of the pier so as to provide a 5-ft. clearance beyond the neat lines of the pier on all sides. Four horizontal frames and one inclined frame were provided in the original plan. The inclined frame was to be set at an angle approximately parallel to the slope of the rock floor as far down as the rock surface would permit, and was designed to resist the pressure on the outer or river face wall. Later, a second inclined frame, between the bottom frame and the lowest horizontal frame, was added to the design to reduce the stresses on the lower struts. Simple hydrostatic pressure was the basis for estimating the stresses, the influence of the silt and mud being disregarded. Each strut consisted of a pair of timbers, breaking joints and separated by the width of the vertical 12 by 12-in. posts, with 12-in. filler blocks at each joint which also served as splice material. The general arrangement is shown in Fig. 3, which is a view of some of the work on Contract HRB-2.[2] The pair of timbers in each strut varied in size from 6 by 12 in. at the top to 12 by 14 in. for the lower sets. At the intersections, vertical posts were framed between the timbers to provide a continuous member from the lower to the uppermost frame so that each frame could be built near the water surface and so that it could be sunk by the weight of the succeeding frame added above.

The rangers or walers were 24-in. and 30-in. I-beams, supported by timber brackets bolted to the under side of the strut and extending beyond the ends of the strut for the width of the beam. Where the steel walers overlapped at the corners they were secured by steel brackets or knee-braces bolted to

[2] *Proceedings*, Am. Soc. C. E., October, 1932, p. 1362.

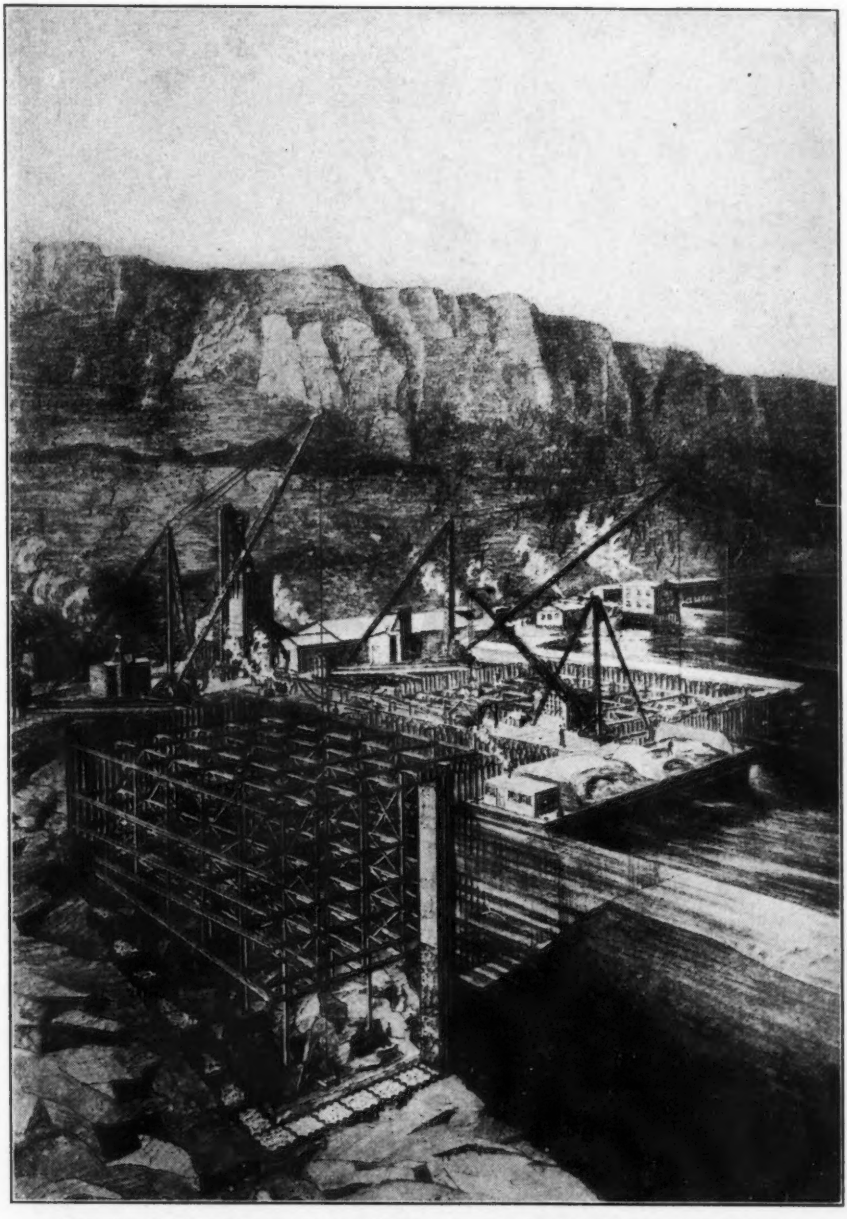

FIG. 2.—PERSPECTIVE OF COFFER-DAMS FOR NEW JERSEY TOWER FOUNDATIONS.

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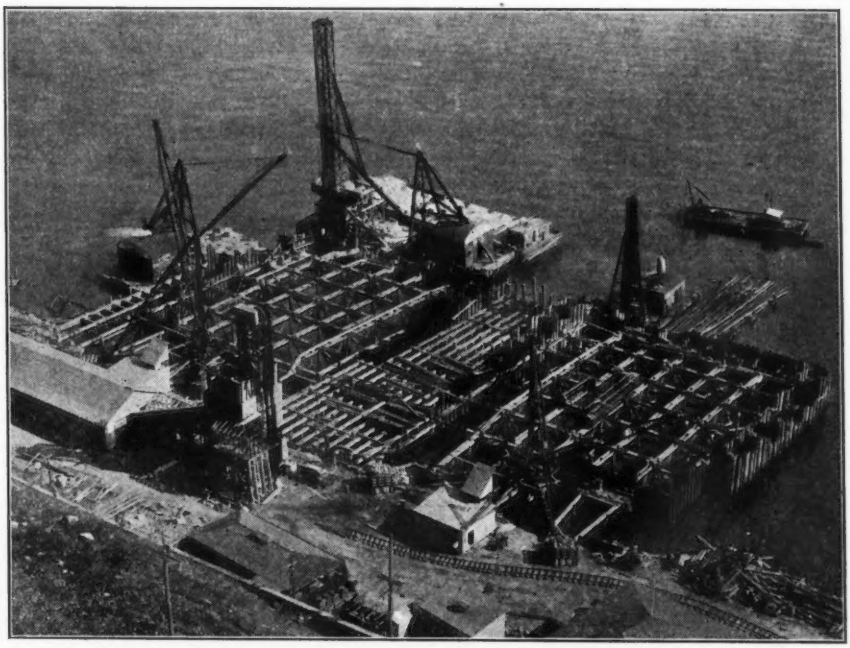

FIG. 3.—TIMBER BRACING AND STEEL SHEET-PILING IN COFFER-DAMS, NEW JERSEY TOWER.

FIG. 4.—PREPARING FOUNDATION ROCK IN SOUTH COFFER-DAM ALONG WESTERLY WALL, NEW JERSEY TOWER.

UNIV. OF MICH.

the inner flanges of the waler beams. Five struts were used longitudinally and five transversely, dividing each coffer-dam area into thirty-six pockets of about 16 by 18 ft. This arrangement provided unusually free working space for hoisting muck and lowering concrete. Diagonal timber bracing was used in the vertical plane from the top to the bottom set of each alternate row of struts to provide trussing to support the dead weight of the timber and such working platforms and miscellaneous loads as might be placed on the bracing.

A double wall of sheet-piling was used along the east or river face and inshore for about four-fifths of the distance on the up-stream and down-stream ends. For the remainder of the distance to the inshore wall, and along that wall, a single line of sheeting was used. The two lines of sheeting forming the double wall were spaced 7 ft. 10 in. apart and were connected by cross-diaphragm walls forming pockets or cells 10½ ft. long.

The dredging operation was started early in May, 1927, and more than 75 000 cu. yd. of the silt overlying the rock were removed in 20 days. Several large basalt boulders and an old rock-filled crib were encountered along the land side of the south coffer-dam which made it necessary to employ divers to drill and blast these obstructions. About 10 days were required for their removal.

After the dredging was completed, guide-piles were driven to support and control the sinking of the frames. These braced clusters of guide-piles were located so as to fall within the pockets formed by the bracing frames. As each frame was completed, it was lowered to such a level as to permit assembling the next frame just above the surface of the water, the vertical 12 by 12-in. posts serving to space the frames in the vertical position. As the lower frame approached the bottom of the dredged basin, further work by divers was required to clear up overlying riprap, boulders, and logs that interfered with the lowering of the bracing to its final elevation. The assembling and sinking of the frames were completed so that the sheet-pile driving could be started in July, 1927.

The steel sheet-piles weighed 32.9 lb. per ft. and varied in length from 40 ft. along the land side to 85 ft. along the deepest part of the river face. A total of 1 558 tons of sheeting was required for the two coffer-dams. The sheeting was set up against the steel waler of the bracing by means of a derrick-boat and driven with steam hammers both with and without swinging leads. (Fig. 3.) To avoid mistaking large boulders for the bed-rock, the final elevations of the points of the sheeting were checked carefully and compared with the elevation of the rock surface as determined by the final core borings. The greatest difficulty in seating the points of the piles against or into the rock was met in driving the sheeting on the shore side and on the ends on account of some interference from boulders and possibly some remains of the crib at the southwest corner of the south coffer-dam. The tendency of the tops of the longer piles to creep ahead also caused some difficulty which was overcome by providing a specially fabricated wedge-shaped pile which was inserted to bring the edges parallel at the position where the closing pile was placed.

At first, all pockets formed by the double walls of the coffer-dam were to be filled with silt. However, in order to strengthen the wall below the bottom waler, concrete was used in the pockets along the river face and for a distance of four pockets on each end of the coffer-dam. This concrete extended from the rock surface up to the level of the lowest waler. To minimize the possibility that the exterior silt pressure might distort the vertical alignment of the sheeting, the silt on the inside was excavated in alternate pockets. The major portion of it was removed by a clam-shell bucket, but near the bottom, the remaining silt was taken out by suction on a centrifugal pump, guided by a diver to make certain that the rock surface was thoroughly cleaned of all silt and boulders and to insure close contact of the concrete seal in these pockets with the bed-rock. While the silt was being pumped, the water in the pockets was carefully maintained at a level, to avoid adding hydrostatic pressure to that of the exterior silt. Concrete was placed in these pockets with extreme care by a 1-yd., bottom-dump bucket, equipped with a canvas cover to prevent wash over the top surface as the bucket was lowered. After making certain that the bucket was on the bottom, it was raised slowly as it was dumped. In this manner the concrete was brought to an elevation slightly above the lower walers after which a mixture of river mud and sand was introduced as back-fill. The undredged pockets and the space behind the single pile wall along the land side were filled with similar material, as had been originally planned.

When the unwatering operation was begun, the precaution was taken to hold the water level just below each set of braces until all piles that were not bearing directly against the steel waler were blocked out, thus preventing distortion of the sheeting as the water was lowered. The coffer-dam was unwatered, without difficulty, by means of two vertical, submerged, electrically-driven centrifugal pumps, one an 8-in. and the other a 12-in. pump. When the water had been lowered to the level of the ground on the high side, the excavation of the remaining overlying material was begun. The softer materials were dug out with clam-shell buckets and the rough rock surface was cleaned by means of high-pressure water jets. Disintegrated rock was loosened with crow-bars and wedges and loaded by hand into hoisting skips. Little blasting was necessary.

The rock floor consisted of alternate layers of sandy shale and sandstone with bedding planes sloping toward the shore at an inclination of 15 to 20 degrees.[2] The foundation bed was prepared by removing the shale, leaving benches of the harder sandstone with vertical steps of several feet (Fig. 4). When the foundation rock along the shore side (amounting to about one-third of the total area) had been prepared, concrete was poured to an elevation sufficient to embed the lower two sets of bracing (Fig. 5). The concrete was carried beyond the neat lines of the pier into direct contact with the steel sheeting for a depth of 5 ft., to provide further security to the coffer-dam. This program was repeated around the ends and until the river wall of the coffer-dam had been sealed securely with concrete. The operations were then extended over the entire area, care being taken to stagger the construction

[2] *Proceedings*, Am. Soc. C. E., August, 1932, Fig. 12, p. 996.

FIG. 5.—CONCRETE ALONG SHORE SIDE OF SOUTH COFFER-DAM AND PREPARATION OF MIDDLE THIRD OF FOUNDATION, NEW JERSEY TOWER.

FIG. 6.—TREMIE CONCRETE AND FOUNDATION ROCK AT NORTHWEST CORNER OF NORTH COFFER-DAM, NEW JERSEY TOWER.

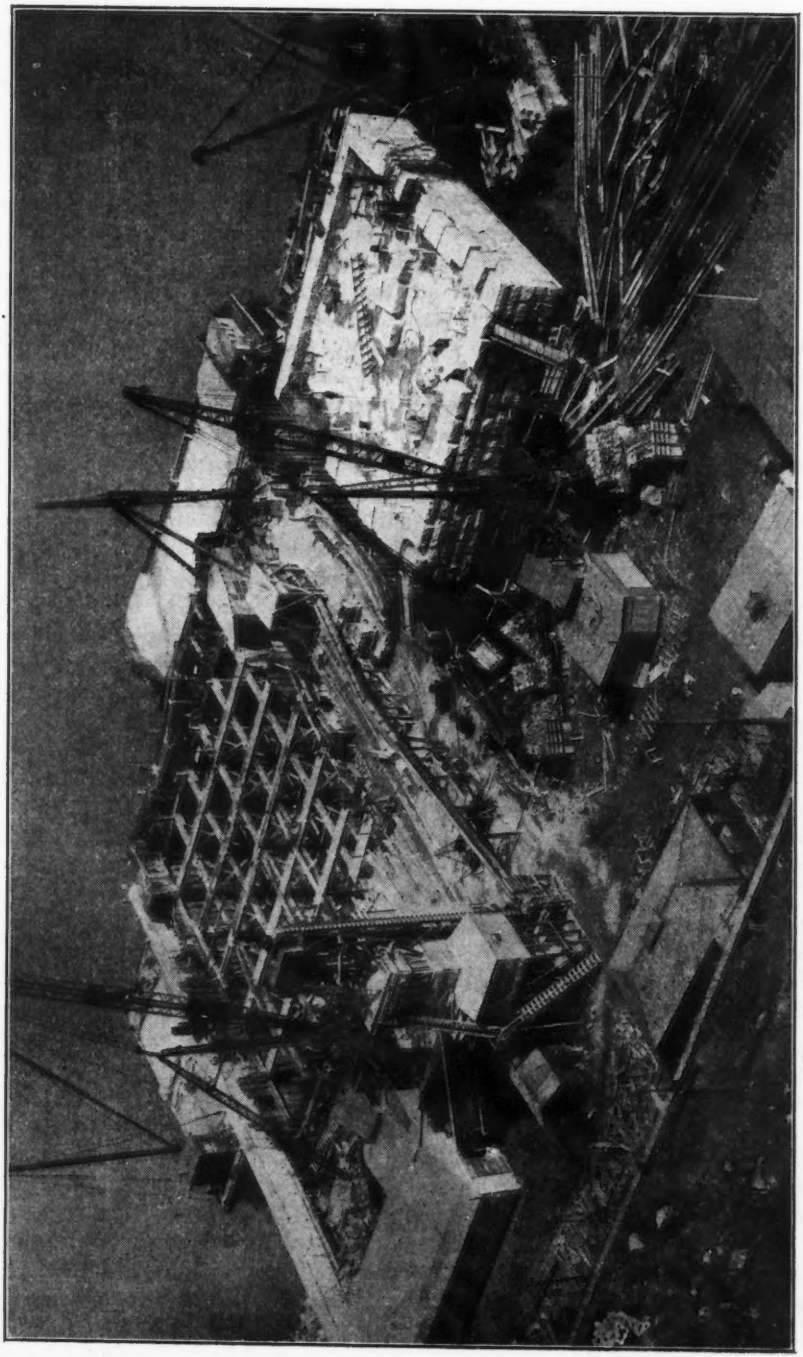

FIG. 7.—LAYOUT OF PLANT FOR CONSTRUCTION OF NEW JERSEY TOWER FOUNDATIONS.

joints. When the concrete had been poured to the elevation of the bottom of the granite facing, the coffer-dam was strutted against the concrete, and the set of braces next above was removed. As soon as the masonry had been built above high tide, the coffer-dam was flooded, the top set of bracing was removed, and the masonry was completed without obstructions. After the bottom had been sealed and while the masonry was being built up, these coffer-dams, more than 100 ft. square, were so tight that a 4-in. centrifugal pump, operating only part time, was sufficient to take care of all leakage.

When the foundation bed along the inshore edge of the north coffer-dam had been prepared and a portion of the concrete over this area had been placed, a blow-in occurred in a single line of sheeting at the up-stream, inshore corner, on the morning of December 23, 1927. The fissured rock failed to carry the reaction at the point of the sheeting, with the result that the sheeting was bent inward around the bottom waler as a fulcrum. Unfortunately, three men were killed; had the accident occurred at a later time in the day, the loss of life would have been much greater. The bent sheeting was removed, and this section was re-driven with a double wall. Since the rock foundation had already been prepared for concrete, it was possible to place a form under water and to remove the silt that had been carried in through the break. After this further preparation of the bottom by divers, sufficient concrete was placed by tremie in this corner to secure the toe of the new sheeting and to support the entire inshore, and part of the up-stream, wall of the coffer-dam (Fig. 6). The coffer-dam was then pumped out again and the work carried to completion, following substantially the same program. The rock along the outboard or river face of this coffer-dam was eroded to a level 78 ft. below high tide. Two additional walers were placed along this river face and strutted back to the rock.

All the construction plant on this contract (HRB-2) was electrically driven. Its layout is shown on Fig. 7. Concrete aggregate was delivered by water and unloaded on to a belt conveyor by a stiff-leg derrick, on a timber dock built between the two coffer-dams. The belt conveyor discharged into bins above a 2-yd. concrete mixer erected on shore. The batching was by weight and an automatic timing device controlled the minimum time of mixing. The cement was conveyed from the water-front on a belt supported by a trestle built along the up-stream end of the north coffer-dam. This belt, running back from the water-front, discharged on to a second belt running at right angles, which passed through the center line of a cement storage shed and up to the level of the mixer charging floor, so that cement was normally moved from the barge upon which it was delivered directly to the charging floor. When the concrete plant was not operating, the cement from the barge could be taken from the belt and stacked in the shed for storage, later to be loaded on the same belt for transportation to the mixer. The greater proportion of the cement was moved directly to the mixer without storage, resulting in considerable saving to the contractor. The mixed concrete was handled in 1-yd., bottom-dump buckets moved over a narrow-gauge track, in 2-car trains to within reach of the derricks that were located to serve each

of the coffer-dams. These foundations were completed in April, 1928, and immediately afterward the erection of the bearings for the steel tower was begun. It is believed that open coffer-dams of any such area have not previously been carried to so great a depth.

New Jersey Anchorage

One of the conditions to be met in the construction of the New Jersey anchorage was to insure its completion in time for the wire-spinning operations on the main cables when the towers and the New York anchorage should be ready for this operation. Accordingly, the contract for the excavation of the New Jersey approach and anchorage tunnels (Contract HRB-3) provided that the work at the face of the Palisades and for a short distance back of the anchorage tunnels should be completed by October 1, 1928, while for the remaining excavation of the cut through the Palisades, completion was not required until January 1, 1930.

This anchorage consists essentially of two inclined tunnels or shafts, spaced 106 ft. apart, one for each of the two pairs of cables. In these inclined shafts the steel grillage girders and eye-bars to which the cable strands attach are embedded in concrete. These shafts were made wedge-shaped, with a cross-section of approximately 44 by 55 ft. at the bottom and 26 by 37 ft. at the top, the center line having an inclination of 37° 18′ from the horizontal. The anchorage girders are located 240 ft. along the axis of the tunnel below the turning point of the cables, which is a short distance below the upper roadway level.

The roof of the tunnel was excavated with an arch varying in radius from 25 to 30 ft. Provision was made in the contract for supporting this roof with timbering if it proved necessary. The contractor was to furnish and set whatever temporary timbering or shoring might be required for properly and safely carrying on the work. He was to be compensated on the basis of 50% of the actual cost of the material used and left in place after the excavation was completed. Although the roof span varied from 26 ft. at the mouth to 44 ft. at the bottom of the tunnel, it developed that no timbering of any kind was necessary.

Below the level of the future lower roadway, the excavation took the form of a rectangular pit, 47 ft. deep, 30 ft. wide, and 65 ft. long, which was necessary to provide sufficient working space during the operation of spinning the cable wire. The distance from the floor of the anchorage pits to the bottom of the tunnel excavation was 150 ft. along the axis of the tunnel.

The cable saddles at the New Jersey anchorage were supported by blocks of concrete, 26 ft. high and 27 by 69 ft. in plan, resting on the rock floor of the cut just forward of each of the anchorage pits (Fig. 8). These concrete masses also serve to anchor the lateral system in the horizontal plane of the wind chords which will function as the upper chords of the stiffening trusses when the lower deck is added to the existing structure. In the early design studies these anchorage saddles were supported on short rocker bents which would take care of the changes in cable length between the eye-bar heads and cable saddles. However, due to the close proximity of the anchorage for the

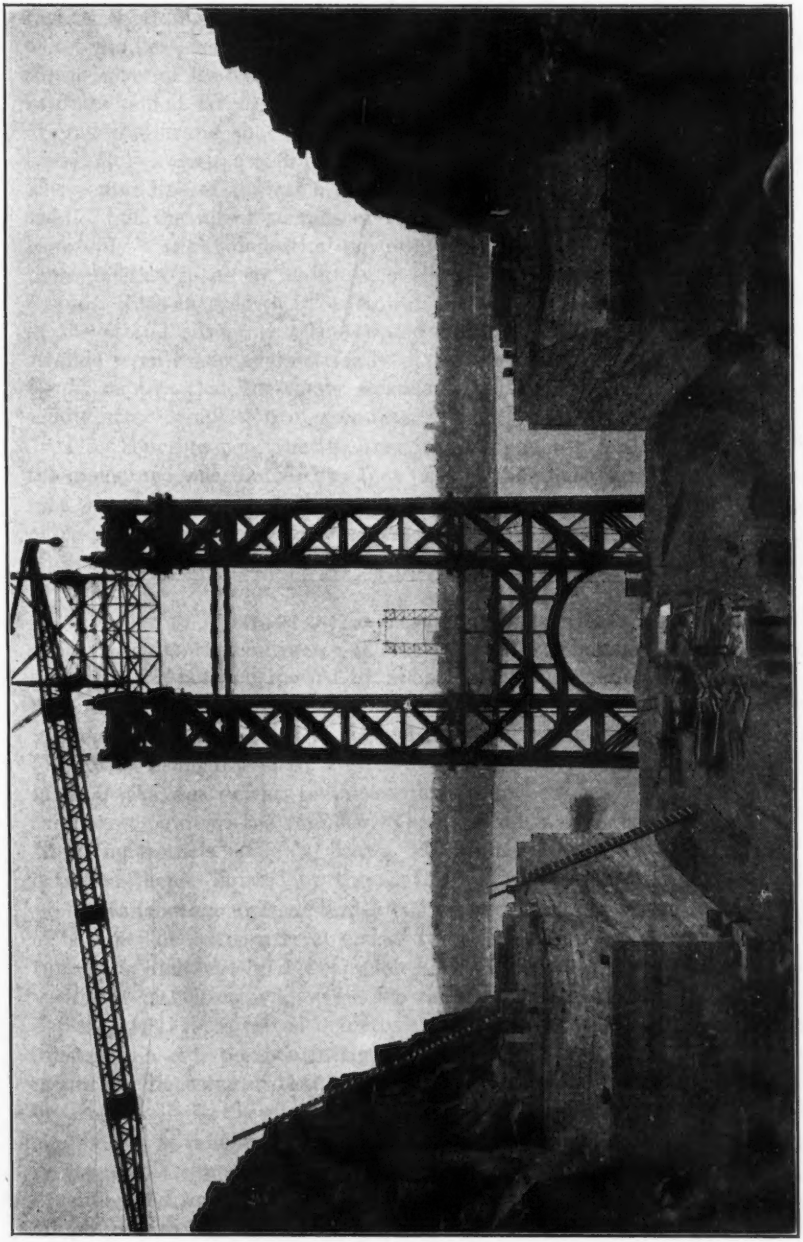

FIG. 8.—NEW JERSEY ANCHORAGE BLOCKS IN OPEN CUT.

lateral system of the floor steel, it was found better to combine the support for these saddles and the lateral anchorage into a single concrete mass. These saddles also serve as splay castings, providing for the forces incidental to the splay of the sixty-one strands from the hexagonal shape at the saddles into the much larger rectangular shape where the individual strands pass around the strand shoes at the eye-bar heads and are connected to the eye-bars. Roller nests are provided to take care of the movements at this point. The center line of the cable changes direction by about 10° at the anchorage saddle, but the upper strands of the cable leading to the upper strand shoes have only a slight deflection downward. This was accomplished by locating these saddles so as to maintain a slight downward deflection of the upper strands. This insured their bearing in the saddles and avoided the disturbance of the strand adjustment that would result if the splay or deflection of the strands were made symmetrically about the center line of the cable, which would require that the upper strands be pulled down and restrained in the saddle after completion of spinning.

This concrete construction forming the support for the cable saddles of the anchorage was included in that part of the anchorage excavation contract that was to be completed by October 1, 1928, in order to avoid further division of responsibilty between the contractors and to insure that all parts of the New Jersey anchorage would be ready for the cable contractor in the fall of 1928.

In order to excavate the anchorage tunnels with the approach cut and anchorage pits, the contractor (Contract HRB-3) planned to excavate the tunnel from the bottom upward by adopting mining methods. A shaft located beyond the north rim of the cut and opposite the back or west end of the tunnels was sunk to a level somewhat below the lowest point of the tunnels (Fig. 9). From this shaft a cross-drift sufficiently wide to accommodate two mine tracks was driven to the southward to pass behind and slightly below the lowest point of the tunnels. This location was made such that the muck from the tunnels could be drawn off through chutes directly into the mine cars and thence hoisted up the shaft for disposal.

The anchorage tunnels and pits involved the excavation of 23 800 cu. yd. of basalt. The east or river end of the approach cut overlying the anchorage tunnels, which was to be completed by October 1, 1928, required the excavation of 99 600 cu. yd., while the remainder of the cut to be completed by January 1, 1930, involved a further excavation of 97 900 cu. yd. Before submitting his bid, the contractor had determined that advantageous arrangements for disposing of this 22 300 cu. yd. of excavated material could be made by crushing it at the site and selling it as crushed rock to dealers in concrete aggregates, withholding only sufficient quantity for the concrete to be placed in the saddle supports and anchorage tunnels. Accordingly, an efficient rock-crushing and screening plant was erected north of the rim of the cut through which the rock from the tunnels, pits, and approach cut was passed and delivered at the crusher bins to a local dealer who trucked it to a near-by site for storage.

It was the contractor's original intention to dig the tunnels by stoping methods, drawing off only sufficient muck to take care of the swell of the excavated material and leaving the remainder as a footing and support for the stoping drills which were to take out the full section of the tunnels as they progressed upward to the roof. It was found, however, that the slope of the tunnel was too great for the muck to remain in a stable pile on the floor and that it did not afford a sufficiently safe and secure support for the workmen and the drills. The procedure was changed accordingly after the

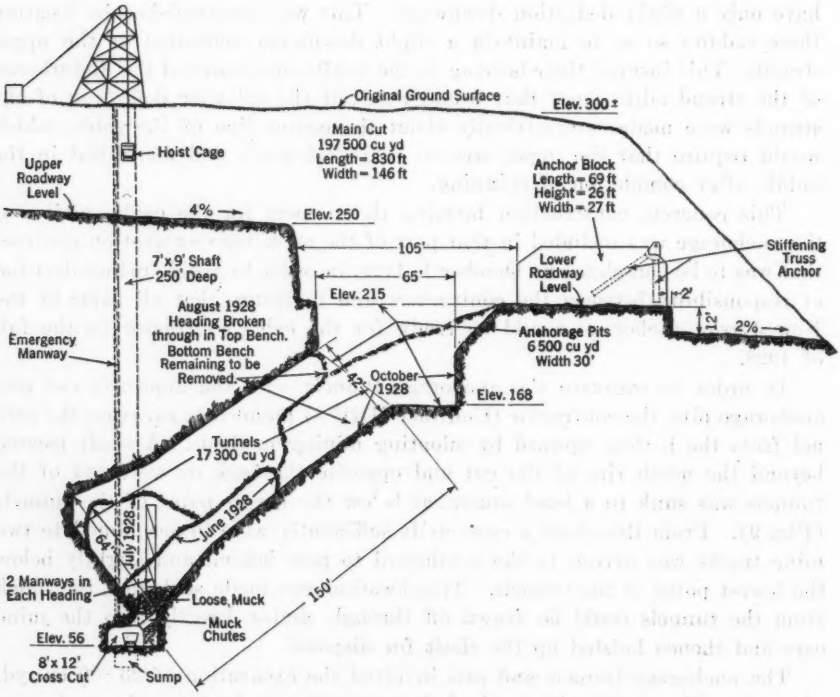

FIG. 9.—DIAGRAM OF ANCHORAGE TUNNEL EXCAVATION.

excavation for the full section had progressed about half-way up the length of the tunnels and from that point a full crown drift was taken out up to where the tunnel was holed through into the anchorage pits. The bench remaining below this crown drift was then shot, largely by rounds drilled down from the bench, some face drilling being done along the limiting planes of the tunnel. The program or schedule of drilling was varied to meet the shape, size, and position of that part of the heading to be blasted. In general, the shots included from 80 to 100 holes. The work on both tunnels progressed simultaneously, which made it possible to employ the muckers and drillers alternately in the other tunnel during the time necessary to ventilate the working chamber after each blast.

During the period when the full heading was taken out and the excess muck allowed to accumulate in the bottom of the tunnels, access to the working chamber was maintained by two timbered manways into each tunnel. These manways, in addition to the necessary ladders, also provided space for the air lines, water pipes, and the ventilating pipes. It was necessary, of course, to carry the drill steel, dynamite, and other supplies through these manways. Walkways were built to overcome some of the difficulties from the sliding of the muck and to provide more convenient access from the top of the manways to the headings. These were supported on transverse cables which were anchored into the side walls of the tunnel.

The handling of the long drill steel up through the manways and over the walkways to the heading proved an expensive and tedious procedure, so that after the heading had advanced a little more than one-third the way up the tunnel, an 8-in. vertical well-drill hole was sunk from the bottom of the cut through to intersect each of the tunnels. The drill steel and dynamite could then be lowered into the tunnels from the surface by light, air-operated hoists. This greatly expedited the work and reduced the cost of transporting supplies to the working face. These 8-in. holes also supplemented the ventilation duct which was a 14-in. spiral riveted pipe leading down the hoisting shaft and up through the manways.

The survey lines and grade elevations in the tunnels required considerable tedious instrument work. Lines and grade elevations were projected down the hoisting shafts through the cross-drifts and up the manways into the tunnels. When the tunnels had been "holed through" into the anchorage pits, it was found that the instrument work checked within a few hundredths of a foot.

After the main excavation had been completed, the anchorage pits and tunnels were trimmed to clear all encroachments within the area required for the anchorage steel, and preparations were made for placing the anchorage girders and eye-bars. The relatively small amount of seepage into the pits and tunnels was drained to a sump at the foot of the hoisting shaft and pumped from that point. Rubble and concrete masonry was placed to seal off the openings from the bottom of the tunnels into the cross-drift, and concrete footings were placed for the support of the anchorage girders.

The accurate location and alignment of the anchorage girders and eye-bars was greatly facilitated by providing steel falsework which was embedded with the anchorage steel. This arrangement is well illustrated by Fig. 10, which is a view of the New York anchorage. This falsework was fabricated accurately and checked so that a minimum of instrument work in locating a few of the main members assured the correct location of all the anchorage steel. All the girders and the bottom three tiers of the eye-bars were put in placed before any concrete was poured.

In order to reduce the effect of the clearance between the pin and the pin-holes in the eye-bars and girders, the eye-bars were held against the pin in the direction in which later the cable pull would be applied. This was accomplished by means of the U-bolts shown in Fig. 11, which passed around the

head of each eye-bar and forced the head face of each pin-hole in close contact with the pin, thus eliminating the back lash that would occur if the bars were allowed to slack down so that the bar face of each pin-hole rested against the pin.

There were four lengths or tiers of eye-bars between the anchorage girders and the cable strand shoes. Only three tiers were placed and embedded in concrete previous to the wire-spinning operations. The eye-bars of the fourth tier were placed as the spinning progressed, thereby furnishing more working room during the spinning operation and making it possible to embed the upper or fourth tier of eye-bars after substantially all the dead load stress had been applied to them.

As a precaution against the shrinkage of concrete, which would permit ground-water to reach the anchorage steel, mats of reinforcing bars were provided on the six faces of the embedding concrete; that is, parallel to the side walls, roof, floor, and lower end and upper end of the anchorage tunnels. This reinforcement consisted of $\frac{3}{4}$-in. rods, spaced 12 in. in both directions. Construction joints in the embedding concrete were avoided by laying out a program that permitted concrete to be poured continuously.

Stop-waters were provided across the construction joint between the concrete embedding the lower three lengths of eye-bars and that placed about two years later around the upper eye-bars carrying the dead load stress of the erected bridge. These stop-waters consisted of a line of $\frac{1}{2}$-in. steel plates around the four faces of the anchorage mass, having one-half their width embedded in the original concrete and the other half projecting so as to be embedded in the final concrete. During the cable-spinning operation, the anchorage pits were drained by pumping from the construction shaft, into which the seepage flowed through pipes leading down through the embedding concrete into the cross-drift and shaft.

The lining of the anchorage pits was omitted until after all the spinning operations had been completed and the suspended structure erected, but before the concrete deck had been built over the anchorage area. Then it was anchored to the rock walls by 1-in. deformed bars spaced 5 ft. in each direction and anchored into holes drilled 3 ft. into the rock. The walls themselves were reinforced near the face with a mat of $\frac{3}{4}$-in. rods, spaced 12 in. in both directions. In addition, blind drains were provided along the water-bearing seams and brought through openings at the bottom of the wall to reduce the building up of hydrostatic pressure back of these lining walls. After the pits had been lined with concrete, the drainage was cut off at the bottom of the anchorage pits and the small quantity of seepage water was removed from the bottom of these pits into a drainage system at the level of the lower deck by sump pumps equipped with automatic float switches. Although the bottom of the anchorage pits is about 125 ft. below the general surface of the top of the Palisades, it has been found that a very small quantity of seepage water comes in through the lining. A heavy concrete cap was placed over the top of the construction shaft, but the cross-drift and shaft were not back-filled.

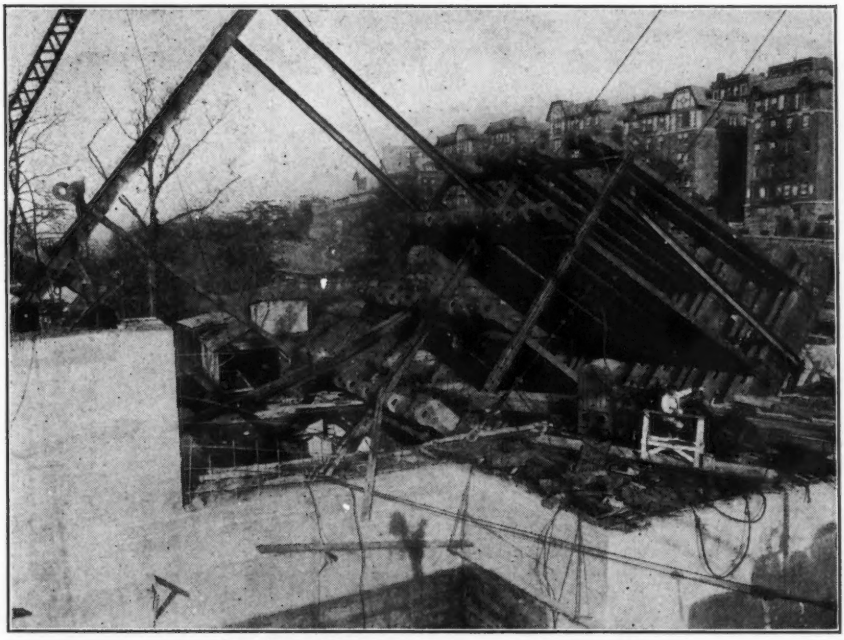

FIG. 10.—NEW YORK ANCHORAGE STEELWORK.

FIG. 11.—U-BOLTS AND COUPLINGS ON EYE-BARS.

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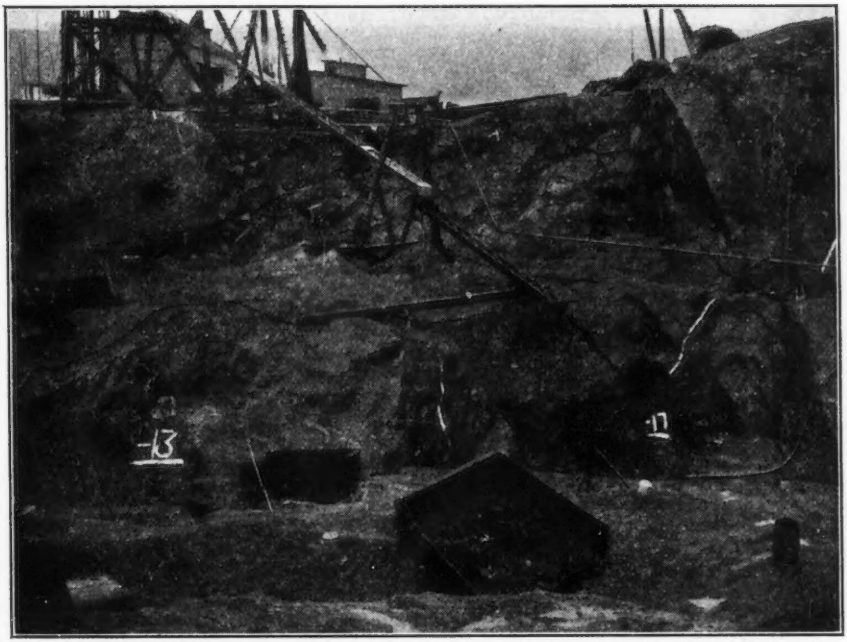

FIG. 12.—NEW YORK TOWER FOUNDATION LOOKING NORTHWEST IN SOUTH PIER AREA.

FIG. 13.—ROCK AT SITE OF NEW YORK ANCHORAGE.

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New York Tower Foundations

The site of the New York tower foundation is on Fort Washington Point opposite 178th Street, in Manhattan. Here, the typical Manhattan schist rises to an average elevation of 10 to 12 ft. above mean high water, with only two small inlets where the rock surface falls below the level of high tide. The preliminary borings showed sound rock, but persistent reports, based on soundings, that this rocky promontory was undercut by the river, and the sharp slope of the rock to a considerable depth, made additional exploratory core borings desirable. These further borings clearly indicated a distinct fault in the rock structure about 200 ft. east of the site, which was apparent from the valley behind the knoll forming the promontory. However, there was no indication of any objectional rock structure under the site, and inclined holes down the river slope proved to be in sound rock for a considerable distance on the river side of the pier. The elevation of the seams in the rock cores showed that the bedding planes sloped toward the river.

The elevation of the top of the pier is the same as on the New Jersey side, and provisions were made for the same dimensions of the finished pier. However, because of certain delays that occurred in procuring the right of way in Fort Washington Park, it was decided to omit the granite facing temporarily, in order to save the time necessary for selecting, quarrying, and cutting the granite, and thereby to advance the date on which the erection of the steel tower could be started in the spring of 1928. The granite facing on this pier was placed in the fall of 1932 and a 7-ft. iron picket fence was built around the top of the coping to prevent trespassing on the structure.

The New York tower foundation, therefore, consisted of two rectangular concrete piers each approximately 76 ft. by 84 ft. 6 in. at the tower base, with footings 80 ft. by 88 ft. 6 in., to include the granite facing. The top surface was reinforced with two mats of 1-in. square rods spaced at varying distances from 10 to 18 in., center to center, in both directions, and a single mat of similar spacing on each face. The faces were built with 2-in. horizontal offsets spaced 2 ft. apart. Hooked 1-in. rods, spaced 2 ft. 8 in. vertically and 10 to 18 in. horizontally, were used to provide a bond between the concrete backing and the granite.

The plans called for the rock to be excavated to Elevation — 11 in the south pier and to Elevation — 6 in the north pier, which plane it was believed would pass well below any surface seams and would provide an ample berm or shoulder of rock along the river face of the pier. The inlets, where the proposed excavation was below the river surface, were readily cut off by a short section of wooden sheet-piling along the south face of the south pier and by a rock dike backed with clay or silt along the north face of the north pier. When the excavation was well advanced, a clay-filled seam extending under substantially the entire area of the south pier was encountered. This seam was from a few inches in thickness to scarcely more than a bedding plane. The slope of the bedding planes toward the river was also somewhat more than had been expected from the core borings. It was accordingly determined to remove the rock to a considerably greater depth. This judg-

ment and decisions in all similar cases was passed upon by Charles P. Berkey, M. Am. Soc. C. E. The final excavation was extended down into sound rock at an average elevation of — 15 in the south pier and to an average elevation of — 5 in the north pier. The quality of the rock may be judged by reference to Fig. 12.

In order to insure the completion of these foundations in time for the tower erection at the earliest practicable date the concrete was mixed in a plant located on a barge moored alongside the site. Since the quantity of concrete involved was relatively small as compared with the anchorage mass, this made no material difference in the cost to the contractor and enabled him to have ample time to set up the more elaborate plant used for handling and mixing the concrete that went into the anchorage.

New York Anchorage

The New York anchorage site is in Fort Washington Park just west of Riverside Drive, opposite the city block between 178th and 179th Streets, Manhattan. The location of Riverside Drive placed certain limitations upon the length of the New York side span. To have located the anchorage east of Riverside Drive would have resulted in a side span slightly longer than desirable or compatible with the design of the towers. The longer side span would have subjected the tower to considerable more deflection under a varying live load than is the case with the shorter side span. The selection of the site for the anchorage west of Riverside Drive resulted in a somewhat shorter side span than has been customary in the proportioning of long-span suspension bridges; but it had certain advantages, which have been clearly stated in a previous paper.[5] The outcropping ledges of rock (see Fig. 13) and the preliminary borings supported the earlier decision that suitable foundations for the anchorage structure were available without excessive stripping of overburden or excavation of rock.

The anchorage structure as designed consists of a mass of concrete approximately 200 ft. in the dimension transverse to the bridge axis and 300 ft. in the dimension along the bridge axis, rising to a height of 85 ft. above Riverside Drive and approximately 120 ft. above the low point in the rock, which was about midway of the length of the anchorage. The anchorage is not a solid rectangular mass; only the rear or easterly third is practically solid with the two wings extending to the west, the extreme ends of which constitute the buttresses for the support of the anchorage cable saddles.

Because of the great dimensions of this mass, with its resulting shrinking during the process of cooling and setting, three expansion keyways were provided; one longitudinal to the center line dividing the structure into north and south halves and the other two transverse to the center line and located approximately at the two-third's point forward from the rear end of the structure (see Figs. 14 and 15). These keyways, 7 ft. wide and extending from the foundation rock to the top of the structure, were left open above Elevation 100 until all the concrete had been placed. The longitudinal

[5] "George Washington Bridge; General Conception and Development of Design," by O. H. Ammann, M. Am. Soc. C. E., *Proceedings*, Am. Soc. C. E., August, 1932, p. 1016.

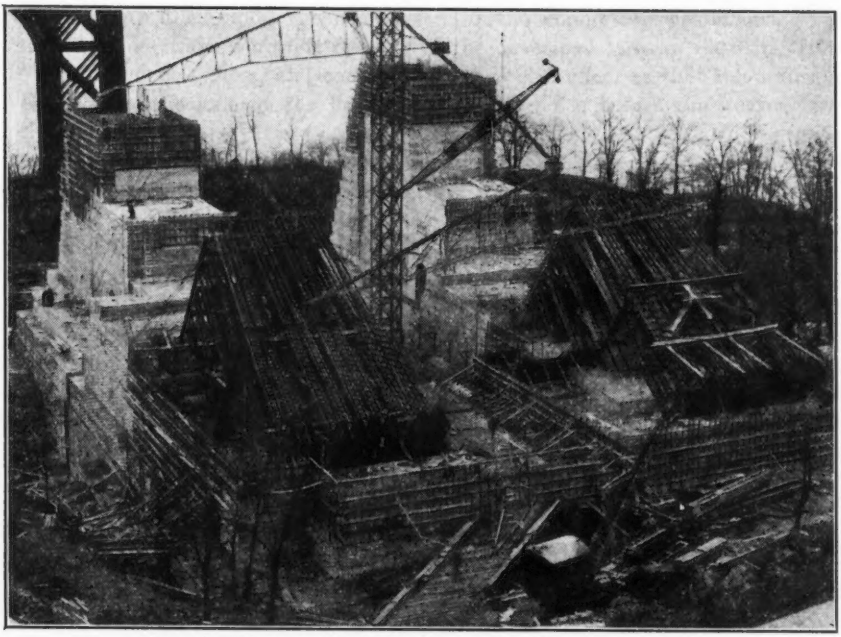

FIG. 14.—EMBEDDING STEELWORK IN NEW YORK ANCHORAGE.

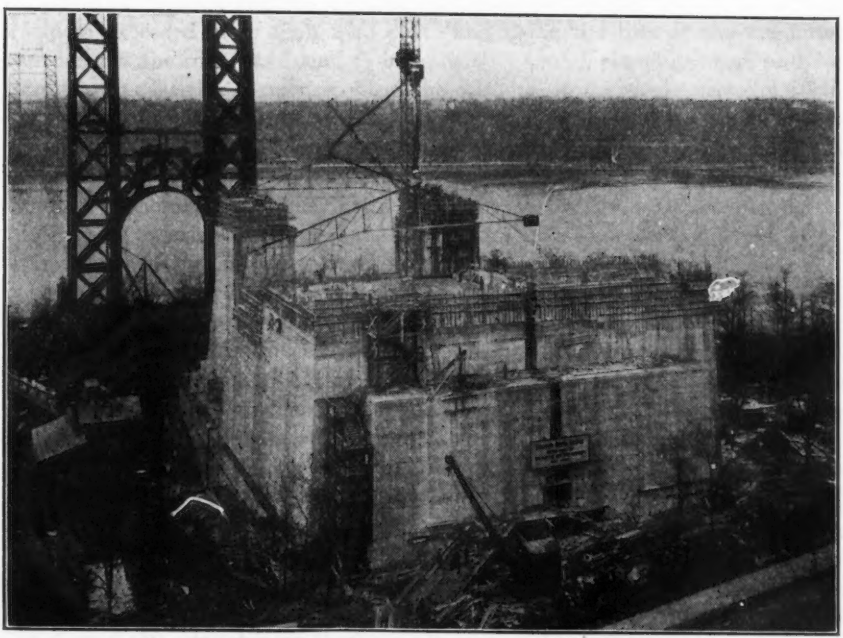

FIG. 15.—NEW YORK ANCHORAGE APPROACHING COMPLETION.

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keyway was filled upon the completion of the concreting operations. The transverse keyways were left open until the dead load stresses resulting from the cables, floor system, and concrete deck, were in effect so that the deformations along the axis of the bridge could take place before the keyway was closed.

The concrete that embedded the upper tiers or links of eye-bars was also omitted until the dead load was acting on the cables. This concrete and that for filling the two transverse keyways were placed by the contractor for the roadway pavement after all that pavement had been put in place.

The anchorage girders embedded in this mass extend about 25 ft. below the rock line. In the early studies and on later drawings for this anchorage structure, provision was made for certain drainage tunnels to be excavated into the rock on the high side of the anchorage girder pit excavation with the idea of intercepting at least a portion of the ground-water and thereby reducing the water pressure against the embedding concrete. A more intimate knowledge of the rock structure at this particular site, as developed by the final borings, made it evident that thoroughly satisfactory results would be obtained by driving a single drainage tunnel through along the low point in the rock, which happened to be located at the transverse center line of the structure. Fortunately, the rock floor continued at a low elevation to the south, so that it was possible to drain this tunnel by gravity without rock excavation, thereby materially lowering the ground-water pressure on the entire mass.

The overburden and the disintegrated mica schist that had to be stripped seldom exceeded more than 2 to 6 ft., and along the line of the anchorage girder pits the rock was found to be unusually sound, closed-grained, and free from water-bearing seams, so that the wisdom of the decision to omit the drainage tunnels was substantiated thoroughly by the conditions encountered during the excavation. In the northwest corner, however, it was found that the rock was badly disintegrated and since the buttresses at the westerly end of the anchorage carry the thrust from the anchorage cable saddles, it was necessary to excavate somewhat deeper than had been contemplated at the time the contract drawings were prepared.

The anchorage structure is without reinforcement except at the surface, along all exterior faces, where it consists of 1-in. square bars on 18-in. centers both ways. On horizontal surfaces and in certain parts of the structure somewhat more highly stressed, the bars were placed on 12-in. centers. In addition to this surface reinforcement, all the exterior faces were studded with hooked 1-in. square rods, spaced about 4 ft. each way. These rods provide for bond between the concrete and the granite face that is to be added at a later date. A reinforcing mat was also provided in the surfaces in contact with the rock in the anchorage girder pits to give additional protection against shrinkage cracks in the concrete, which might allow the ground-water to reach the anchorage girders. The precaution was taken of pouring this concrete lining of the anchorage girder pits as a monolith. A grillage of rods was also placed beneath the anchorage cable saddles.

Recognizing the time element involved in obtaining the eye-bars and anchorage girders for this structure, provision was made in the contract for the cables, for that contractor to deliver the anchorage steel at the site without expense to the anchorage contractor. Provision was also made in the design for the steel falsework to support the anchorage material previous to its embedment (Fig. 10). As described for the New Jersey anchorage there was also a requirement that the eye-bars be held against the pin as shown in Fig. 11.

The concrete for the anchorage was designed for a compressive strength of 2 000 lb. per sq. in. in 28 days and was proportioned by the water-cement ratio method (as was the practice on the other foundation structures), with thoroughly modern weighing batchers.

The 110 000 cu. yd. of concrete involved in this structure justified a careful study of the plant to be selected for its accomplishment, and no doubt the successful bidder on Contract HRB-4 was aided materially in the prices he was able to bid by his confidence in the belt-conveyor plant that he adopted. In general, the writer understood that at the completion of this contract the cost of moving the cement and concrete aggregate by belt conveyor from the dock about 1 000 ft. distant to the structure was less than the cost of unloading the material from the barges to the dock. Because such belt-conveyor plants for moving the cement and aggregates from the dock to the structure and the mixed concrete from the central mixing plant to the foot of the hoisting tower were at that time somewhat unique, additional space will be devoted to a more detailed description of the plant used in constructing this anchorage.

The plant designed for transporting, mixing, and placing the concrete in the New York anchorage produced at times as much as 1 200 cu. yd. in a 16-hour day. The entire mass, approximately 110 000 cu. yd., was put in place in 5½ months, or somewhat more than 2 months ahead of the schedule. This was accomplished by a modern, electrically-operated mixing plant on the south side of the anchorage, approximately opposite the transverse center line. Fig. 16 shows the plant layout.

Sand and gravel were unloaded from barges by stiff-leg derricks and placed in separate piles over a 6 by 6-ft. wooden tunnel, 300 ft. long, which paralleled the dock. Hopper gates in the roof of the tunnel under the piles of aggregates allowed the latter to be fed to a belt conveyor in the tunnel. Sand and gravel were alternately dumped upon this belt as needed. The belt through the tunnel discharged directly upon a pair of belts in series which carried the aggregates up the incline of Fort Washington Park to the concrete plant. An interesting feature of the belts is the comparatively steep grade negotiated, which reached a maximum of 32% over a part of the length. This system had a capacity of more than 300 tons per hour and discharged into a 300-ton storage bin at the top of the mixing plant.

Cement was handled in bags directly from barges to the dock by cranes on the barges. A continuous belt from the dock to the mixing plant was located directly under the second and third sand and gravel belt. The bags

were placed on this belt by hand. As will be observed from Fig. 16 the belt passed through a cement storage house near the water-front. Thus, the cement could be taken off for storage or carried directly to the mixing plant as required. The cement belt had a capacity of 50 tons per hour and discharged on to a covered platform at the charging level of the mixing plant. The overhead bins at the mixing plant had a capacity of 100 tons of sand and

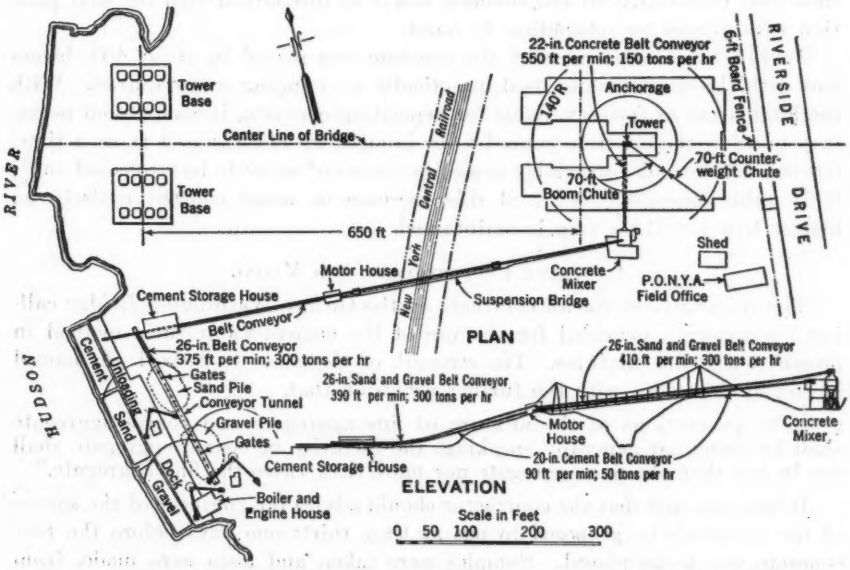

FIG. 16.—PLANT LAYOUT FOR TRANSPORTING, MIXING, AND PLACING CONCRETE, NEW YORK ANCHORAGE.

200 tons of gravel. Under normal operation the cement was delivered to the charging pipes at the rate of using in order to avoid re-handling. Only the excess due to short shot-downs was piled. A storage of 2 000 bags was provided on the covered platform at the charging level.

Two 1-yd., electrically operated, automatic time mixers were charged by a single weighing batcher plant. These mixers dumped the concrete into a common receiving hopper which, in turn, discharged to a belt conveyor, 125 ft. long, operating through the temporary tunnel through the south wing of the anchorage structure. This belt discharged into a receiving hopper at the base of a 260-ft. tower at the center of the anchorage.

The steel hoisting tower was equipped with two chuting systems, each consisting of a 75-ft. boom chute, from which was suspended a 75-ft. counterweight chute. In order that the desired low-slump concrete could be transported without clogging, 14-in. chutes were used to provide for easy flow. Concrete was hoisted to the receiving hoppers at the top of the boom chutes by a 36-cu. ft. roller hoist bucket, arranged to be dumped into either chute. This bucket traveled at a speed of 380 ft. per min., and was operated by a hoist driven by a 100-h. p. electric motor. The same hoist could also be used

to handle the chute booms and the lifting frames of the chute system when re-adjustments in level were being made. The discharge end of the counterweight chute was provided with a small hopper and a short "elephant trunk" to insure that concrete was always discharged in a vertical direction to avoid the segregation which would follow if it was allowed to be discharged at any angle much from the vertical. The flexibility of the chuting system was such that practically all the concrete could be discharged into its final position with almost no rehandling by hand.

In this large mass most of the concrete was placed in about 4-ft. layers and with the consistencies used, practically no tamping was required. With the large areas of form available for depositing concrete, it was seldom necessary to allow the concrete poured to be brought up more than 4 ft. at a time. Greater depth would invariably cause the excess of water to be "squeezed out"; it is quite impossible to avoid this increase in water content entirely no matter how low the slump is maintained.

Concrete Proportioning and Mixing

The specifications for all contracts on the George Washington Bridge calling for concrete, provided for the use of the cement-water ratio method in proportioning the mixtures. The strength of the concrete desired was named in the specifications with the further provision that,

"The proportions of the mixture of fine aggregate and coarse aggregate shall be such that generally speaking, the quantity of coarse aggregate shall not be less than the fine aggregate nor more than twice the fine aggregate."

It was required that the contractor should advise the engineer of the source of the materials he proposed to use at least thirty-one days before the first concrete was to be placed. Samples were taken and tests were made, from which it was determined what the mixture should be, the cement-water ratio having been stated in the specifications.

From time to time throughout the work under each contract, samples were taken from the concrete, both at the mixer and from the forms, and made into cylinders for tests of the actual strength and densities obtained. Adjustments in the mixtures were made from time to time based on the variations in the fineness modulus of the concrete aggregates. The water content of the sand was checked from time to time in order to maintain control of the total water entering each batch. The test cylinders used were 8 by 16 in. made by the Field Staff of the Construction Division, in accordance with the standard methods of the American Society for Testing Materials for making and storing concrete specimens in the field (Serial Designation, *C*31-21). The contractor was required to supply adequate space and facilities at or near the mixing plant for making the cylinders. Occasional check tests were made by the Field Staff of the Port Authority's laboratory in Jersey City, N. J.

Contracts involving concrete provided for the furnishing of the cement by the contractor to the Port Authority at the contractor's bid price per barrel, so that the Port Authority retained complete freedom of judgment as to any variations in the mixtures that might prove desirable. The equity as between the Port Authority and the contractor is of course only maintained

within the reasonable limits commonly experienced with the aggregates available in the commercial markets tributary to the bridge construction.

It was provided that the consistency or workability of the mix should be measured by means of the slump test made in accordance with the tentative specifications of the American Society for Testing Materials (Serial Designation, *D*138-26*T*). The ordinary slumps required were between 2 and 3 in., but, of course, there were cases in which departures were made from these normal consistencies. As an explanatory phrase to the contractors it was stated in the specifications that if, at any time, the contractor desired a more workable mix than that obtained under the 3-in. slump, he must obtain the result by reducing the total quantity of aggregate per batch and not by the addition or diminution of water.

The specifications provided in general that all concrete should be produced in a central mixing plant and that previous to the beginning of the work, the contractor must secure the approval of the engineer for the methods and apparatus he proposed to use. Except for the New Jersey anchorage all the concrete mixing plants on the foundations that are the subject of this paper, were provided with weighing batchers and time-control devices on the discharge lever of the mixer. For concrete used in the approaches to the bridge and for the New Jersey anchorage pre-weighed batches were hauled to the job in batch compartment trucks and mixed in standard paving mixers. In a few cases, for isolated operations at times when some of the main plants were not in operation, ready mixed concrete was used. Little difficulty was experienced in getting satisfactory results with the ready mixed product, but the precaution was taken to maintain an inspector at the plant during the time that the product was being delivered to the bridge.

In the case of the concrete used in the tunnels of the New Jersey anchorage a brief study was given to the possibilities of getting a dense, impervious concrete without spending too much for strength. There was no escape, however, from the generally accepted principle that imperviousness and strength follow along substantially parallel curves, so that considerable strength was built into the concrete in the New Jersey anchorage for the sole purpose of obtaining an impervious concrete where the strength was not essential.

In the case of the concrete used on the deck of the main bridge, the aggregates were hauled to the job in bulk, batched at the New York anchorage, and hoisted to the deck level. The dry and unmixed aggregates were then transported over the deck to the mixer alongside the point of deposit, so that the mixed concrete could be discharged directly into place in the deck slab. This was done in order to maintain a more precise control of the water content, which was so vitally necessary in producing a pavement of satisfactory quality.

The strengths obtained on the concrete used in the four principal foundations described in this paper are shown in Table 1.

In general, the results obtained on the various contracts for the George Washington Bridge fully demonstrate the economies possible by the use of the cement-water ratio method in proportioning concrete. There were a few

instances, however, which demonstrate most forcibly the dangers involved where some unthinking individual attempts to follow the formulas blindly when the results clearly indicate that some blunder in the computations has been made. The writer is thoroughly convinced that still further economies

TABLE 1.—STRENGTHS OF TEST CYLINDERS, FOUNDATIONS OF THE GEORGE WASHINGTON BRIDGE

Age	NEW JERSEY TOWER		NEW YORK TOWER		NEW YORK ANCHORAGE		NEW JERSEY ANCHORAGE	
	Number of cylinders	Breaking strength, in pounds per square inch	Number of cylinders	Breaking strength, in pounds per square inch	Number of cylinders	Breaking strength, in pounds per square inch	Number of cylinders	Breaking strength, in pounds per square inch
28 days	112	2 200	55	2 990	306	2 480	35	2 960
3 months	32	2 940	14	3 130	93	3 010	15	3 670
6 months	29	2 790	9	4 120	83	3 190	14	4 250
1 year	4	3 230	..	.....	27	3 730	6	4 210
2 years	5	3 760	4	5 320	19	3 420	5	5 210
3 years	3	3 810	1	5 500	17	3 550	4	4 450
4 years	2	4 100	1	5 780	..	.....	..	.....

can be accomplished as the methods of handling concrete from the mixer to its position in the forms are improved so that workability is obtained with a still greater saving in the cement content and with a resulting strength well beyond that demanded by the needs of the structure under construction.

PERSONNEL

The work of the construction of the New Jersey pier and tower foundations, known as Contract HRB-2, was performed by the Silas Mason Company. Francis Donaldson was Chief Engineer and H. L. Meyer, Superintendent for the Company. The excavation of the New Jersey anchorage was done by Foley Brothers, Incorporated, Contract HRB-3, for which, Carl Swenson was General Manager, John Holmes, Superintendent, and G. H. Wilsey, M. Am. Soc. C. E., Chief Engineer.

Both the New York tower foundation and the New York anchorage were combined in one contract, HRB-4, the Arthur McMullen Company, Contractor. For the contractor the work was under the general direction of E. P. Palmer, M. Am. Soc. C. E.; H. B. Gates, Assoc. M. Am. Soc. C. E., was Chief Engineer.

Of the Port Authority Staff, in charge of the work, F. E. Cudworth, M. Am. Soc. C. E., was Resident Engineer, and A. H. Morrill, Assoc. M. Am. Soc. C. E., Assistant Resident Engineer, of the New Jersey section during the period of construction of the New Jersey tower foundations and the excavation of the New Jersey anchorage, and R. H. Hoppen was Resident Engineer, and R. F. Wheadon, Assoc. M. Am. Soc. C. E., Assistant Resident Engineer, of the New York section during the construction of the New York tower piers and New York anchorage.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

GEORGE WASHINGTON BRIDGE CONSTRUCTION OF THE STEEL SUPERSTRUCTURE

BY E. W. BOWDEN,[1] AND H. R. SEELY,[2]
ASSOCIATE MEMBERS, AM. SOC. C. E.

SYNOPSIS

The erection of the towers, the cables, and the floor steel of the George Washington Bridge is described in this paper. The general characteristics of the steel superstructure are enumerated briefly, after which a detailed treatment is given of the erection methods and of the equipment. Each of the three distinct parts of the work—the towers, the cables, and the floor steel—is discussed separately in the order in which the work was done.

The erection of the large quantities justified unusual attention to detail and refinement in equipment. The entire project proceeded according to a carefully prepared construction program which was substantially adhered to throughout. The equipment used and methods pursued are, therefore, of special interest.

INTRODUCTION

Two contracts for the steelwork for the main structure of the bridge—one for the towers and floor steel combined, and the other for the wire cables, suspenders, and anchorage steelwork—were executed November 4, 1927. In all, fourteen bids were received for the work from five different contracting companies.

Bids had been called for on both wire cable and eye-bar cable designs. Under both designs bidders were permitted to submit proposals for the entire steel structure, or for the towers and floor steel as one contract, and the cables, suspenders, and anchorage steelwork as another. An opportunity was given to base the bids upon alternate programs of erection—"simultaneous" or

NOTE.—Discussion on this paper will be closed in April, 1933, *Proceedings*.

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"successive." Simultaneous erection contemplated the construction of the four cables complete before work was begun on the erection of the floor system. Successive erection contemplated the construction of one of each pair of cables prior to the erection of the floor steel and the construction of the remaining cables immediately thereafter.

For the wire cable design, alternate arrangements were also offered. Cables could be built with the two cables of each pair arranged side by side or one above the other.

The lowest combination of bids received was that for the towers and floor steel as one contract and the wire cable construction as another; the latter bid was based on cables arranged side by side to be built by the program of successive erection. By mutual agreement, this was changed later to the program of simultaneous erection.

The contract for the towers and floor involved furnishing and erecting approximately 40 200 tons of structural steel in the towers and approximately 19 100 tons in the floor system. The contract for the cables and anchorage steelwork involved furnishing and erecting approximately 28 300 tons of cable wire, 170 200 lin ft of $2\frac{7}{8}$-in. steel wire suspender rope, about 6 000 tons of structural steel and castings, and the furnishing, but not the erection, of about 2 200 tons of structural steel (eye-bars, pins, and girders) for the New York anchorage.

Description

The steel superstructure has been fully described in previous papers of this series dealing with design. However, brief descriptions of the towers, cables, and floor steel are included with this paper, in order that the reader may have clearly in mind the features essential to an understanding of the erection procedure.

The Towers.—Each steel tower[3] is essentially an assembly of four rigidly connected bents. A pair of columns composes each leg of a bent, the inside column being vertical in a plane at right angles to the axis of the bridge, while the outside column slopes in toward the top with a batter of $\frac{3}{16}$ in. per ft. The columns forming a leg are spaced 47 ft 6 in. from center to center at the pedestals and 38 ft 3 in. at the tower top while a constant distance of 106 ft between centers is maintained throughout the height of the inside columns of the two legs. All bents batter toward the transverse axis of the tower, the outside bents, $\frac{3}{16}$ in. per ft and the inside bents $\frac{1}{16}$ in. per ft. The spacing between the bents at the tower base—18 ft 8 in. from center to center—is thus reduced to 12 ft 6 in. at the tower top.

The towers are 559 ft 6 in. high from the top of the pier (15 ft above mean high water) to the top of the tower at the base of the cable saddle grillage. Vertically, the towers are divided into twelve panels which have different dimensions for the two towers because the roadway level at the New Jersey tower is 16 ft 6 in. higher than at the New York tower. The panel heights vary from a maximum of 56 ft 6 in. to a minimum of 38 ft 3 in.

[3] "George Washington Bridge: Design of the Towers," by Leon S. Moisseiff, M. Am. Soc. C. E., *Proceedings*, Am. Soc. C. E., December, 1932, p. 1727.

The columns are single-cell box-sections, with webs or extensions projecting at the corners and are of varying sectional areas. The maximum section is found in the inside line of columns in the top panel and is 1 296 sq in. The sectional area of all columns in the lowest panel is 717 sq in. The columns, are spliced immediately above each panel point. They are of silicon steel, but the bracing members are principally of carbon steel.

The Cables.—The four 36-in. cables are in pairs spaced 106 ft. apart; the cables forming a pair are arranged side by side, 9 ft between centers. Each cable contains 26 474 cold-drawn galvanized steel wires, 0.196 in. in diameter, which are grouped into 61 strands of 434 wires, each strand being independently attached at the anchorage to a pair of eye-bars forming the upper links of a chain extending down into the concrete of the anchorage, to terminate at anchor girders.

At the towers and anchorage buttresses, the cables are carried on cast-steel saddles. Those at the towers are of unusual size, $8\frac{1}{2}$ by 28 ft in plan, and weigh approximately 180 tons. Rollers under the saddles at the towers provide for motion due to changes in loading conditions during construction. Rockers care for the more limited motion of the anchorage saddles due to these changes, combined with temperature changes during and after construction.

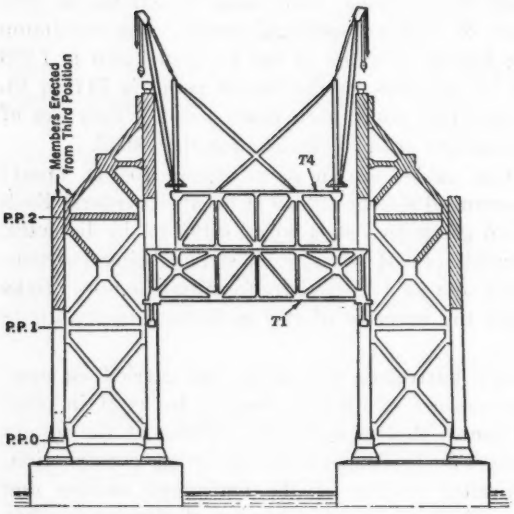
Cast-steel cable bands, bolted together in halves, encircle the cable at each panel point. Two grooves in each band retain the suspender rope loops, thus providing four parts of $2\frac{7}{8}$-in. wire suspender rope from each cable for attachment to the floor-beam at a panel point.

The Floor Steel.—As now in place, the floor system is a single deck only, without stiffening trusses. It is 118 ft in over-all width, providing for a roadway 90 ft wide between outside curbs, the roadway space being divided into three parts by intermediate curbs, the two outside parts only being concreted at the present time. On either side of the roadway are sidewalks for pedestrians.

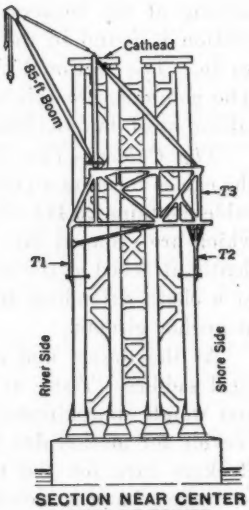
The main transverse floor-beams—plate girders, 10 ft deep and 118 ft long—are spaced at 60-ft intervals and are attached to the socketed ends of the suspender rope groups. Eight longitudinal roadway stringers in each panel, varying in depth from 5 ft 4 in. to 5 ft 8 in., frame into the floor-beams and carry secondary transverse 16-in. I-beams, spaced at 5 ft 2-in. intervals. The main reinforcement of the concrete deck consists of 6-in. bulb-beams, spaced 15 in. from center to center running longitudinally, supported by the secondary transverse beams. A longitudinal beam adjacent to the roadway curb and a fascia stringer support the sidewalk beams, which are spaced at 3 ft 9-in. intervals.

Stress expansion in the roadway and sidewalk is provided at each panel. The longitudinal members framing between the floor-beams are riveted to the floor-beam at one end, but are left free to move at the other.

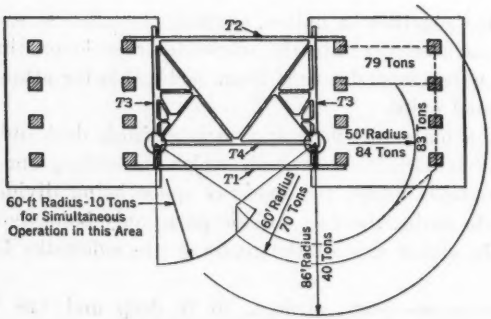
The roadway floor-beams are held rigidly spaced by a continuous wind chord which passes through the floor-beam about 6 ft from each end. The wind chord is designed to become the upper chord member of a light stiffen-



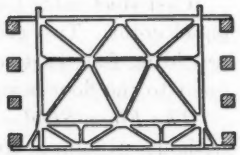
TRAVELER IN THIRD POSITION



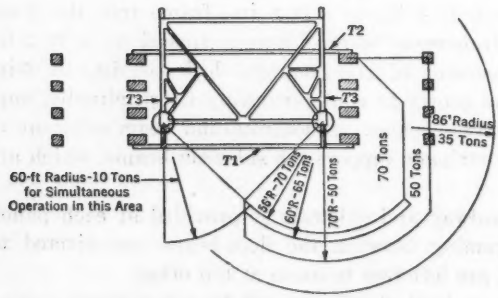
SECTION NEAR CENTER LINE OF TOWER



TRAVELER AT BASE OF TOWER
Derrick Capacities Below Panel Point 6



BRACING AT BOTTOM OF LONGITUDINAL TRUSSES *T3*



TRAVELER AT TOP OF TOWER
Derrick Capacities Above Panel Point 6

FIG. 1.—DETAILS OF TRAVELER FOR TOWER ERECTION.

ing truss system when the lower deck is added at a future time. The gusset-plates for connection with the other members of the stiffening truss are attached to the chord, but are left blank to be drilled when the stiffening trusses are added.

Expansion joints for temperature changes for the floor system are provided at the towers. The wind truss, which consists of continuous lateral bracing in the plane of the wind chord, terminates at the tower in a strut member on the center line of the bridge. The wind reactions are transmitted from this member to the tower steelwork by guides between which the strut floats freely under normal conditions. The stringers in the panel adjacent to the tower are designed to slide freely on the steel framing of the tower floor. At the roadway level, bar-grating expansion joints are provided.

Tower Erection

Erection Equipment.—The conditions peculiar to the towers, especially the wide spacing between the inside column lines and the comparatively large area occupied by the columns composing each leg, led to the design of an erection traveler with two stiff-leg derricks, one for the erection of each tower leg.

The derricks were carried by a steel framework that spanned the opening between the tower legs, as shown in Fig. 1. Two transverse trusses of this framework, *T*1 and *T*2, carried the reactions to the inside columns on the river and shoreward faces of the towers. These trusses were braced together by adjustable struts and cable guys. A pair of longitudinal trusses, *T*3, parallel with, and 13 ft inside, the center lines of the inside column rows, were framed above the transverse trusses and were carried by them. The masts of the stiff-leg derricks were set up on these longitudinal trusses, approximately 12 ft back from the river face of the tower, at the point of framing a transverse connecting truss, *T*4. The transverse and longitudinal trusses carried the reactions of the stiff-legs of the derricks. Bracing members were provided in the upper and lower chord planes of the longitudinal trusses, *T*3.

The batter of the columns at the tower faces introduced a special problem in the traveler design. The distance between the outside faces decreased a varying amount at each panel, the total decrease from the lowest to the highest positions of the traveler amounting to 15 ft 8 in. It was thus necessary to decrease the spacing between the transverse trusses that carried the reactions to the tower columns, at successive positions of the traveler. To meet this condition rollers were provided at the points of bearing of the lower chords of the longitudinal trusses on the upper chord of the shoreward transverse truss. The arrangement is illustrated in Fig. 2. The shoreward transverse truss, *T*2, was thus movable with reference to the remainder of the system. To facilitate and insure the necessary adjustment two 20-ton telescopic jacks were provided. After each jump of the traveler, before proceeding with steel erection, the longitudinal trusses were lifted from the rollers by means of screw-jacks.

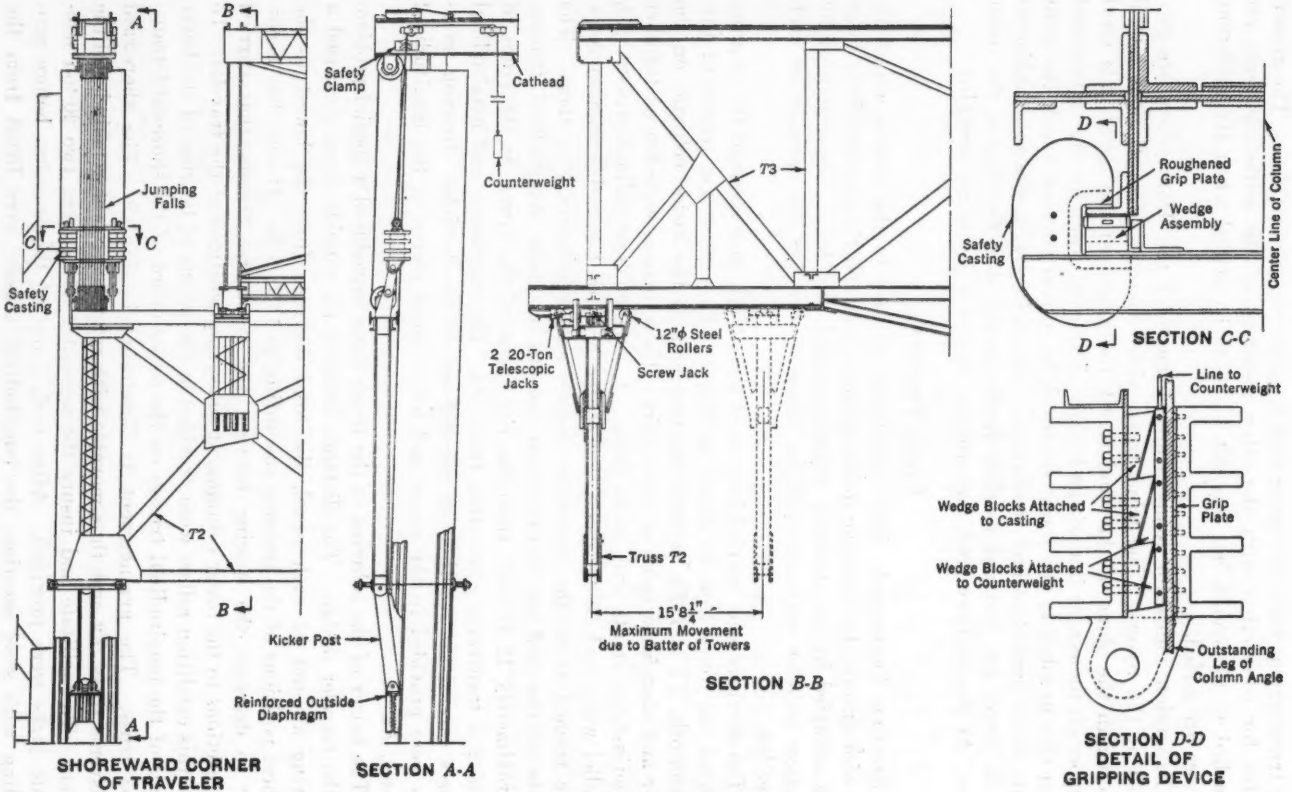


FIG. 2.—TRAVELER DETAILS.

The transverse trusses that spanned between the tower legs were assembled in two stages. As first erected, it was necessary to omit the end posts, a section of the bottom chord, and the end diagonals of each truss, since these members would have fouled the pedestals. Temporary members carried the trusses until the traveler was jumped to a position opposite Panel Point 1. At the New Jersey tower the temporary end members were carried on blocking at the piers. At the New York tower the reactions were carried to rock, near the inside faces of the piers, by means of temporary posts.

The transverse trusses were completed when the traveler was advanced to its first position opposite Panel Point 1, and short kicker posts were added for the purpose of carrying the traveler loads to the tower columns. Throughout their length the columns are stiffened by diaphragms on the inside and between the webs on the outside, at intervals of $6\frac{1}{2}$ to 7 ft. The outside diaphragms, when properly reinforced, were utilized to support the traveler, forming seats for the kicker posts, which were pin-connected to the transverse trusses. An eccentric thrust caused by pulling the post in to engage the shelf angle was overcome by a yoke of channel sections and rods which passed around the column and secured the transverse truss against the column at the pin connection. The total weight of the traveler was approximately 320 tons. The derricks were equipped with 85-ft booms. Their loading capacities are given in Fig. 1.

It was permissible for both booms to handle their maximum loads simultaneously, provided that in so doing neither should encroach upon an area directly in front of the traveler. Within that area the maximum permissible load for either boom was 10 tons if, at the same time, the other boom was handling its maximum load. A boom extension was also provided, good for 15 tons in any position.

Two engine units for the operation of the two derricks were set up on concrete bases at the ground level, about 150 ft shoreward of the tower. Each unit was driven by two 150-hp, 440-volt, three-phase, alternating-current motors, and each unit comprised seven drums—two for the main load falls, two for the boom falls, one for the auxiliary load falls, and two for jumping the traveler. The seven-drum engine was practically a coupling of two engines—one of four drums and one of three drums, each engine with one 150-hp motor.

The boom falls consisted of twenty-two parts of $\frac{7}{8}$-in. cable and the auxiliary load falls of two parts of 1-in. cable. The main load tackle consisted of twelve parts of $\frac{7}{8}$-in. steel cable in a single length of 9 800 ft reeled on two drums so that each end of the line became a lead line capable of a speed of 250 ft per min under a tension of 20 000 lb. When the drums were operated together this had the advantage of double the hoisting speed and better distribution of the hoisting load among the parts. Even with this arrangement, loads hoisted to the top of the tower on the main falls required approximately 17 min for the trip.

Frictional losses were reduced by the use of roller-bearing sheaves for the main load, boom, and auxiliary load falls. Bull-wheels and small electrically operated engines, on the traveler, were used for swinging the booms.

The traveler was "jumped" or advanced from panel to panel by means of four beams of box-section, termed "cat-heads," set up on the tops of the inside column rows. (See Fig. 2.) Each beam was carried by two columns and projected far enough beyond the outside column to support an 18-part 1-in. wire-rope falls above an end of one of the two main transverse trusses. The attached projecting splice material at the tops of the columns made it impossible to rest the beam directly upon the milled ends of the sections. Two sets of plates were framed transversely to the box-section and held it above the projecting column splicing material. The plates were milled and bore upon the milled ends of the column sections. At points where the bearing plates of the "cat-head" were of less thickness than the column web material, shim plates were used to fill the space between column splice-plates; and the bearing plates were then bolted temporarily to the splice-plates of the column.

An interesting feature of the traveler equipment was the safety device provided at each of the four corners of the traveler to prevent it from dropping in case the jumping falls failed during the process of advancing from one position to the next. The device was designed to function by forcing wedge-blocks into contact between the outstanding legs of column angles and steel castings fitting around these angles, the falling motion causing the gripping action. The castings were pin-connected by links to the upper corners of the transverse trusses. Two sets of wedge-blocks worked together; one set was tap-riveted to the casting while the other set was made to follow along during the jumping operation by means of a counterweighted line that passed through sheaves at the "cat-head." The line passed through a safety cable clamp which permitted movement of the counterweighted line in the upward direction only. In case of a reversal of motion, the gripping action of the safety clamp held the line and forced the wedges into contact. Fortunately, the functioning of this safety device was never required.

A satisfactory system of electric signals was devised for the operation of the derricks. A signalman at the point of erection operated a small portable panel of electric push-buttons which controlled signal lights at the engines. For each drum, or set of drums, white and red lights were provided. The engineer hoisted the load or slacked off only while the corresponding indicator bulb remained lighted. This system of signals was especially effective and accident-proof since the maintenance of a light required a continuous pressure on the button in the hands of the signalman and a discontinuance of the signal in effect, whether by purpose or accident, simply stopped all operation.

When designing this equipment the use of a telephone system with head phones for the engineer had been considered, but was decided against in view of the possibility of serious accident that might result through failure of the order to stop the operation in case of incapacity of the signalman or failure of the circuit.

Provisions for Handling Material.—The tower locations are such as to facilitate the delivery of materials by car-float. The piers of the New Jersey

tower are just offshore, with ample depth for mooring car-floats on the riverward side. The steel sections were thus picked directly from the cars and hoisted to position in the tower. Clusters of piles were located north and south of the piers to assist in the manipulation of the car-floats.

The foundation piers of the New York tower are situated on the rocky point of Fort Washington Park, approximately 80 ft back from the water's edge. Here it was necessary to provide docking facilities and a means of handling the materials from the car-floats to a position within reach of the tower traveler. The construction of a substantial temporary unloading dock in front of the tower offered a problem because of the sharply sloping rock that lies bare at that point. A rock-filled crib type of construction was chosen. The cribs were anchored in the rock bank and tied back to the tower foundation. The broken rock-filling was placed to a height 3 ft above high water. When planked over and sheathed on the river face, a very satisfactory mooring wharf was obtained.

A stiff-leg derrick was erected in position to handle the steel from car-floats to traveler derricks. By this means several carloads of material could be unloaded and stored in front of the tower. However, the main point of storage for the materials of both towers and, in fact, the only point of storage for materials for the New Jersey tower, was at the Jersey City, N. J., terminal yards of the railroads, approximately ten miles down the river. Here, the fabricated steel was brought and placed in ground storage to be reloaded on cars and placed on car-floats as needed for erection at the bridge site.

Erection of New Jersey Tower.—The two towers were erected simultaneously, requiring a duplication of equipment and practically a separate field organization, all, however, co-ordinated under one general supervision. In the main, the same method of procedure was followed in the erection of the two towers, minor variations being made to suit the particular conditions encountered. The New Jersey piers were completed several weeks in advance of the New York piers, permitting erection of the steel for the New Jersey tower to be started six weeks earlier. In consideration of the foregoing facts the following description of the erection will outline the procedure at the New Jersey tower, afterward noting the points of variation and the progress on the New York tower.

In the course of the description reference will be required to the columns, bents, and panel points. This reference need cause no confusion because of the extremely simple and practical system of designation used on the work. The bents were identified by numeral and the column rows by letter. From west to east the four bents composing the New Jersey tower were numbered 1, 2, 3, and 4, respectively. The enumeration was continued in the same manner for the New York tower bents, which were 5, 6, 7, and 8. The column rows were lettered from south to north, the column rows, *A* and *B*, comprising the south leg of each tower and the rows, *C* and *D*, the north leg. Panel points were numbered from 0 to 12, with the zero panel point directly above the pedestals. Using this system of designation any point on

either tower could be readily indicated, and in a manner which insured against confusion between the towers. For example, Panel Point 10 of the column at the northeast corner of the New Jersey tower was designated *D*-410 (Column Row *D*; Bent No. 4, and Panel Point 10). The same point of the column at the northeast corner of the New York tower would be *D*-810 (Column Row *D*, Bent No. 8, and Panel Point 10).[4]

Before erecting any tower steel on the New Jersey piers, the traveler was assembled by derrick-boats and stiff-leg derrick, the latter set up on the north pier for the purpose. The contractor began assembling the framework of the traveler on May 29, 1928. The trusses and derricks were erected, the engines installed on the shore near the piers, and the work of reeving was completed, ready for the erection of the tower steel by the latter part of June. The first of the sixteen pedestals were set on June 23, 1928.

The pedestal is a built-up structural member, 14 by 14 ft in plan, and 6 ft 6 in. high. It is anchored to the pier by six 2½-in. anchor-bolts, those for the pedestals of the bent at the river face of the tower embedding 12 ft 4 in. in the concrete masonry, and the remainder of the bolts embedding 5 ft 4 in.

The pedestals for the New Jersey tower were delivered to the site completely assembled. In setting, a uniform bearing was obtained by a carefully prepared system of mortar bedding, as shown in Fig. 3. The concrete of the piers had been finished off to a level approximately 1½ in. lower than the required elevation for the base slabs of the pedestals. Corrugated pipes had also been set at the anchor-bolt locations at the time the concrete was placed.

As a first step, a pair of steel screed rails were set, one at either side of the pedestal location. These were carefully adjusted to grade by level-instrument observations and were fixed in position by quick-setting cement. After placing wooden caps over the corrugated pipes that provided the open holes for the anchor-bolts, a stiff mortar of 1 part sand and 1 part cement, of a consistency approximating putty, was spread over the pedestal area, but was kept back about 8 in. from four accurately adjusted shim-plates, or pads, set to receive the pedestal and to insure exact elevation.

The mortar was reduced to a plane approximately ⅛ in. higher than the final elevation for the base of the pedestal. This leveling process was accomplished by rolling and by striking off with a screed board between the screed rails. The wooden plugs were then withdrawn from the anchor-bolt pipes and the bolts, equipped with pilot nuts, were placed and grouted. Finally, the pedestal was picked and lowered into position and brought to bear upon the four corner pads.

These pads are worthy of a detailed description. A ½-in. steel plate was first set to exact elevation. Upon this was placed an assembly consisting of two bronze plates separated by a steel plate with a gib. A ⅛-in. steel plate was placed in turn on these. Other plates, of various thicknesses, were furnished, such that refinements of 1/32 in. in the adjustment of this corner-pad

[4] *Proceedings*, Am. Soc. C. E., December, 1932, Fig. 9, p. 1745.

FIG. 3.—SETTING PEDESTAL FOR NEW YORK TOWER

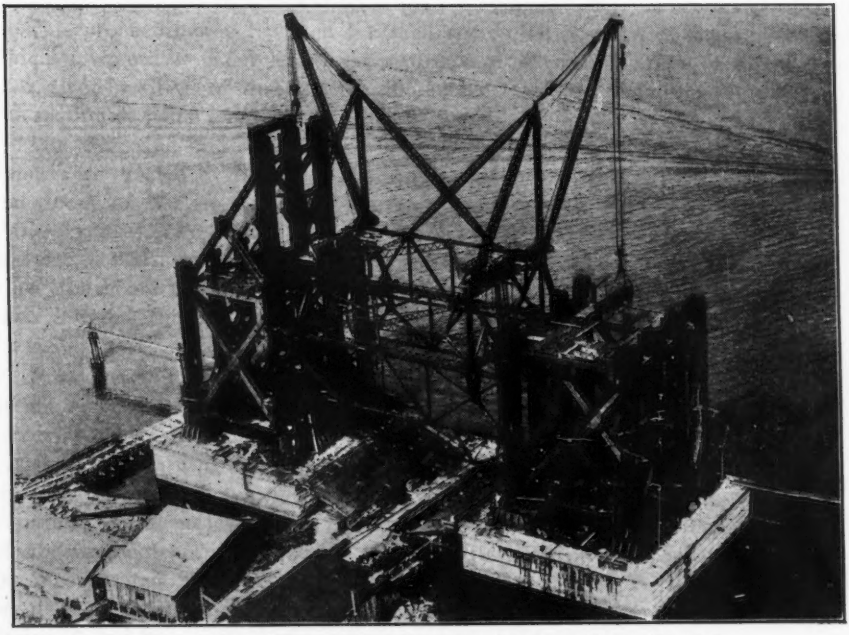


FIG. 4.—NEW JERSEY TOWER TRAVELER OPPOSITE PANEL POINT 1

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assembly was possible. The gib-plate permitted the withdrawal of the entire assembly after the mortar had set.

At first, the mortar was made very stiff, and it was necessary to place other pedestals on top of the one being set, in order to force it down upon the corner pads. Later, a softer mix was used, which permitted easier manipulation. Excellent results were secured by this method of mortar bedding. In one case only was it necessary to pick a pedestal that had been set. An error of $\frac{1}{16}$ in. was detected in its adjustment. When lifted from position it was found that the mortar was entirely free of hard spots, air pockets, or voids. The markings of the planing tool on the slab were clearly visible in the hardened mortar bed. The setting of the pedestals for the New Jersey tower was completed about July 1, 1928.

The traveler then erected all the column sections and bracing of the first panel and also the equipment for advancing the traveler, after which it was "jumped" a half-panel length to a position opposite Panel Point 1 (see Fig. 4) where the transverse trusses of the traveler were completed. The traveler then erected the second panel length of columns forming the two inside rows (the *B* and *C* rows), together with the diagonal members necessary for bracing. The jumping tackle was erected on these inside columns and the traveler was advanced a panel length to a position opposite Panel Point 2.

In this position the traveler erected the second panel length of columns forming the two outside rows (the *A* and *D* rows), the remaining bracing of the second panel and the third panel length of the columns forming the two inside rows (the *B* and *C* rows) with their diagonals.

This procedure was repeated for each successive position of the traveler, which advanced a panel length at a time. At the completion of erection at a given position of the traveler, the inside rows of columns were always one section or panel length in advance of the outside rows.

The arch ring and strut of the lower portal steelwork opposite the third panel were erected by the traveler while it was opposite Panel Point 4. The remainder of the framing for this portal was erected with the traveler at Panel Point 7. To erect the portal steelwork, which was directly below the traveler, the stiff-leg of one of the derricks was disconnected temporarily, to allow the boom to be swung around over the central area. The falls were then lowered through the openings between the members of the traveler frame.

Prior to the erection of the portal steelwork, a pair of horizontal spacing struts were erected temporarily at Panel Point 2, connecting the legs of the tower as shown in Fig. 5. It was intended that these spacing struts should assist in maintaining the proper distance between the separate units. However, they were not sufficiently stiff to permit of much compression. Fortunately, the accuracy of the erected material was such that these spacing struts were not actually required. The struts used in this position were to be incorporated permanently in the structure at Panel Point 3. Similar struts from the top portal steelwork were used in temporary position at Panel Points 6 and 8.

Column sections were delivered for erection with all gusset-plates for struts and diagonals riveted in place. The splice-plates were riveted to the tops of the column sections although the upper rows of rivets were omitted and the splice-angles on the outstanding legs were left loose. Erection of a column section was thus accomplished by lowering or "knifing" in between the splice-plates attached to the column section previously erected. Care was necessary, in order to insure satisfactory joints since, in accordance with the provisions of the specifications, all contact surfaces had received two coats of red lead paint. The entering web usually scraped paint from the inside of the splice-plates, causing an accumulation of the material on the lower contact surface. When the member was within an inch of contact, the lowering of the column section was stopped long enough to permit cleaning the accumulated material from the contact surfaces, by blowing with compressed air. A careful inspection of contact joints was made in all cases, and riveting was permitted only when the opening at point of contact was less than 0.003 in. In a few cases, riveting was delayed until considerable weight of steelwork had been erected above.

Erection of New York Tower.—The New York tower was erected along lines similar to those used in the erection of the New Jersey tower. The first pedestals were set on August 6, 1928. It will be recalled that the pedestals for the New Jersey tower were delivered to the site completely assembled. The pedestals for the New York tower, however, were delivered in two sections for assembly at the site. These were assembled on heavy girder beams with the sections of the base slab in a true plane, and in this position the sections were fitted and riveted together. The method of bedding and adjusting the pedestals by means of corner pads was the same as that for the New Jersey tower.

The erection equipment, the positions during erection, and the methods of procedure were similar in general to those on the New Jersey side, the principal difference came about by reason of the necessity of handling the materials from car-float to tower base by means of an unloading derrick.

It would seem that this use of an additional piece of equipment, together with the personnel required for its operation (a force of about ten men), would cause an increase in the cost of erection. Such was not the case, however, because the unloading derrick was found to effect an actual saving in the time of erection. For one thing, this derrick was of assistance in picking column sections from a horizontal to a vertical position, an operation which otherwise required the use of the second traveler derrick.

The erection of the lower portal steelwork was effected in a slightly different manner from that used in New Jersey, where, it will be recalled, the boom was swung between the stiff-legs so that the falls could be lowered through the truss work of the traveler. In New York, special tackle was hung from the lower part of the traveler frame and used to hoist and drift the various members into position. The latter method appeared to be the more economical in view of the fact that the time required in placing the falls in correct position through the framework more than offset the advantage this method of portal erection seemed to offer.

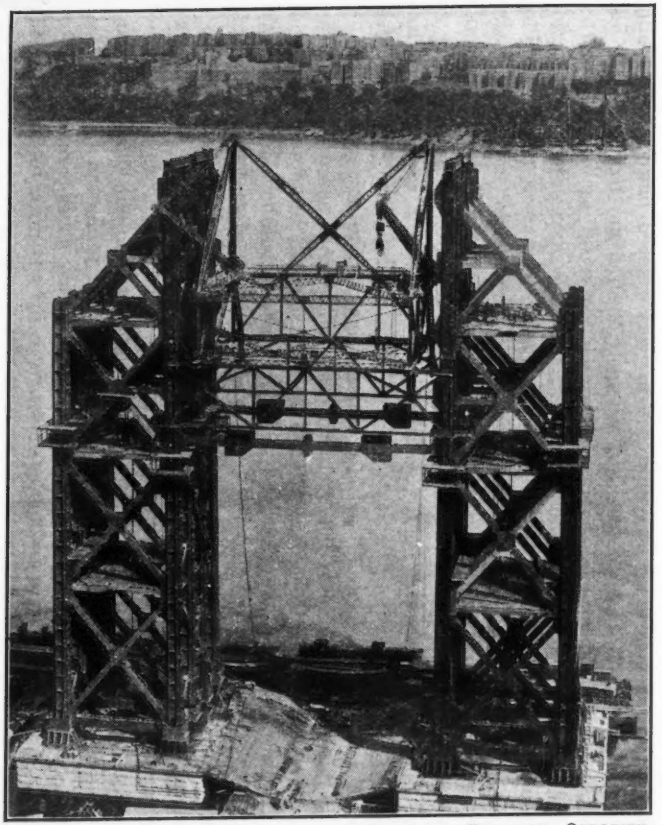

FIG. 5.—ERECTION COMPLETED FOR TRAVELER POSITION OPPOSITE PANEL POINT 3; READY TO ADVANCE TRAVELER TO NEXT POSITION.

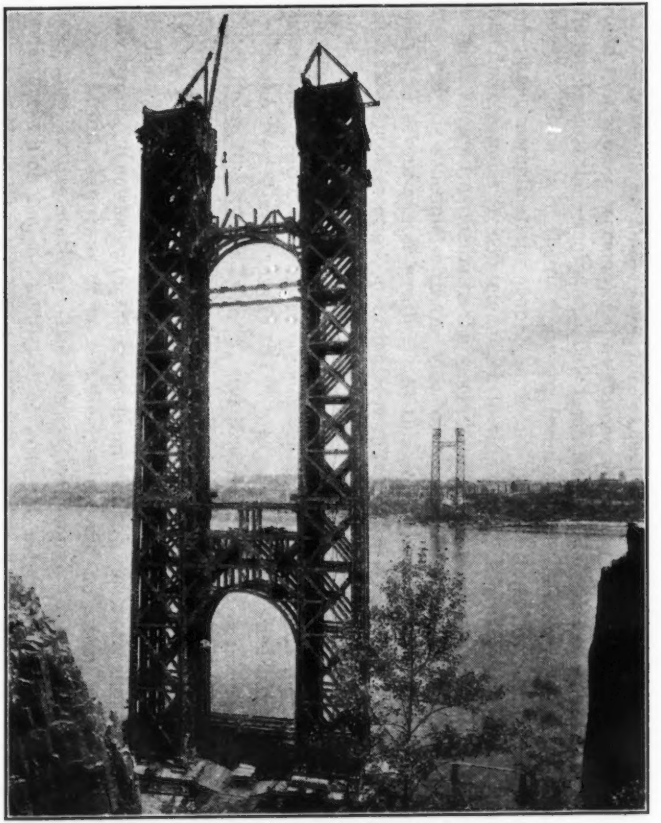

FIG. 6.—ERECTING UPPER PORTAL STEELWORK.

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Observations at Panel Point 10.—The Port Authority specifications provided that the top sections of the columns should be finished in the shop after the erection of the remainder. This permitted the determination of the proper lengths. Because of the deep girders framing into the column sections in the top panel, complications would be found in an attempt to compensate for any discrepancies in column length in this panel. It was decided, therefore, to make such adjustments in the second panel from the top, and the contractor prepared his schedule with the view of reaching Panel Point 10 before the arrival of severe winter weather. At this point the schedule called for a discontinuation of erection operations for the duration of the winter, permitting time for taking measurements to determine the actual relative elevations of the tops of the column sections, and giving an opportunity for finishing the column sections, 10-11, to compensate for the variations found.

In the New Jersey tower, 15 782 tons of structural steel were erected to Panel Point 10 between June 27, and November 1, 1928, a period of approximately eighteen weeks, while, in New York, 15 785 tons were erected between August 6 and November 24, 1928, a period of fifteen weeks.

The observations at Panel Point 10 were made by level instrument, rod, and target. Knowledge of the exact height of the steelwork above the established datum plane was of no practical value, and no attempt was made to determine such elevation. The observations sought only to establish the variations which had accumulated between the columns composing a tower leg.

The readings were somewhat difficult to make because of the attached splice material and because each of the milled surfaces sloped in at least one direction and one-half of them in two directions, due to the column batter. They were taken on the milled surface of each column at four points—two on the transverse and two on the longitudinal center line of the column. When averaged, these readings gave the elevation at the column axis. They were taken separately by two parties on each tower, in order to eliminate personal error, and were made under conditions that would insure, as far as possible, a uniform temperature in all columns; that is, in the early morning or under cloudy conditions. The variations are given in Table 1.

In consideration of the fact that the relative differences in Table 1 were accumulated in the erection of approximately 500 ft of column height, the the small amount of discrepancy found between the columns attests to the high degree of accuracy of the shop work, and the pains taken by the workmen and inspectors in erecting this steel.

Adjustments were made at the shop in the lengths of the columns between Panel Points 10 and 11. The lowest column at Panel Point 10 was assumed to be at plan elevation and to require no correction. All other columns were made shorter by the amount of over-run as indicated.

The usual procedure in preparing the splicing material in the shop was to assemble any given column section with the section to be erected next above it, and to ream the splice material assembled to the columns.

TABLE 1.—VARIATIONS IN COLUMN HEIGHTS AT PANEL POINT 10
(Readings are in $\frac{1}{32}$ in. above the lowest column)

Tower	Column	Variation	Column	Variation	Column	Variation	Column	Variation
New Jersey....	*A1*	+4	*A2*	+6	*A3*	+6	*A4*	+6
	B1	+1	*B2*	+3	*B3*	+4	*B4*	+3
	C1	+4	*C2*	+3	*C3*	+1	*C4*	+4
	D1	0	*D2*	0	*D3*	Lowest	*D4*	+2
New York.....	*A5*	+5	*A6*	+5	*A7*	+4	*A8*	+2
	B5	+7	*B6*	+5	*B7*	+4	*B8*	+5
	C5	+5	*C6*	+5	*C7*	+5	*C8*	+5
	D5	0	*D6*	Lowest	*D7*	+4	*D8*	0

The holes for connecting the splice material to the lower section (9-10) of the columns at Panel Point 10, were handled in the usual way; that is, reamed with splice material assembled to the columns. The holes in the upper half of the splice material were reamed full size ($1\frac{1}{16}$ in. in diameter), to a metal template. The splice-plates and angles were then riveted or bolted for shipment. Later, when the columns in Panel 10-11 had been milled to proper length, the holes for the splice at the bottom of the columns were drilled and reamed full size to the same metal templates. It was intended that the holes should be reamed over-size for larger rivets if found necessary after erection, but such action was unnecessary.

On account of changing the lengths of the columns, some adjustments were required in the connecting bracing members. This was taken care of by varying the position of the groups of open holes in the gusset-plates.

Erection Above Panel Point 10.—Erection was resumed on the New York tower on March 26, 1929, and on the New Jersey tower on April 8, 1929. With the exception of the portal steelwork, the erection of the New York tower was completed to Panel Point 12 during the week of May 7; the last two panels were in place on the New Jersey tower on May 29.

The traveler was then dismantled at Panel Point 12. The procedure for dismantling the traveler and erecting the upper portal steelwork was practically identical for the two towers. A stiff-leg derrick was erected by the traveler on one leg of the tower, and one of the pair of traveler stiff-leg derricks was moved to the other tower leg. These two derricks then dismantled the traveler and erected the members of the upper portal, completing it during the latter part of June. (See Fig. 6.)

The contractor for the towers, as sub-contractor for the cable contract, erected the grillage steel, footbridge ropes, cable saddles, and cable-spinning equipment on the tower tops. Accordingly, when the portal was completed, the second stiff-leg derrick of the traveler, which had been lowered to the ground, was rigged for use at the top of the tower. Sills were added, and its boom was increased 20 ft, to a length of 105 ft. This derrick was set in position on the portal bracing near the center line of the tower and was used to dismantle the derricks on the tower legs. Its capacity was 85 tons at a radius of 60 ft.

Fitting and Riveting.—No difficulty was experienced in the field in erecting and fitting the steelwork because the shop procedure was so carefully planned and executed. The special conditions at Panel Point 10, which made impossible the shop assembly of adjacent column sections, have been considered. Similarly, special conditions at Panel Point 6 made impossible the assembly of the sections in bearing at that point.

Because of the large quantity of steel to be fabricated in the limited time, the part of each tower below Panel Point 6 was fabricated in the shops of a sub-contracting company. The splice material at Panel Point 6 was handled in a manner similar to that described at Panel Point 10, except that the holes for connection to the column sections above Panel Point 6 were drilled to $\frac{15}{16}$ in. and reamed to $1\frac{1}{16}$ in. after erection.

During the course of erection of both towers, instrument observations were made at frequent intervals to check the verticality. Considering the limits of probable error of observation, the alignment was satisfactory in all cases. The accuracy of the shop work had been such that at no time was it necessary to resort to any drifting or "guying" to change the alignment.

Riveting was begun on the New Jersey tower when the first two panels of steelwork had been erected, and on the New York tower, when the first three panels had been erected. Thereafter, the riveting on both towers followed closely behind the erection. Shortly after the discontinuation of erection for the winter months, riveting was likewise stopped. Riveting on the New Jersey tower was discontinued on November 28, 1928, to be resumed on April 8, 1929. On the New York tower the riveting was discontinued on December 8, 1928, to be resumed on April 8, 1929. With the exception of a few scattered points left open because of cable-erection equipment, the riveting was completed on both towers during August.

The rivets were of carbon steel, 1 in. in diameter for the main members, although a few $\frac{7}{8}$-in. rivets were used in the portals; a total of 488 000 field rivets was driven in the New Jersey tower and 498 000 in the New York tower. They were heated by means of soft coal forges. Other methods of heating were tried, such as by electric-heating units and oil forges, but were found to be less satisfactory than the ordinary forges. All riveting was done by pneumatic hammer, and "holding on" was likewise done by a pneumatic tool wherever possible. An air compressor, driven by a 125-hp electric motor and having a piston displacement of 675 cu ft per min of free air, was located near the base of each tower. A pressure of about 100 lb was supplied at the tool.

An average of nine or ten gangs of riveters of four men each were engaged on the work on each tower, although as many as thirteen gangs were employed at one time. The men worked $5\frac{1}{2}$ days per week, barring unfavorable working conditions, and averaged more than 300 rivets per gang per day. Under favorable conditions much higher rivet counts were made. The average number of rivets rejected was 2.7% in New Jersey and 2.0% in New York for the entire period of the riveting.

Special precautions were taken to protect the men engaged in the work of fitting and riveting. Planks, 4 in. in thickness, were used to construct a floor over the entire area of the tower leg at each panel point immediately after erection. This floor remained in place until all rivets in the panel were driven. Scaffolds with timber floor and roof on steel frames were hung on the outside column faces. These were protected with hand-railings and were equipped with ratchet winches so that they could be drawn up a panel length at a time by hand operation. Access to the point of steel erection was usually by means of a large cage hoisted from the tower base by one of the traveler derricks. Access to the various points on the tower during riveting operations was facilitated by an elevator installed in one tower leg. The guides and hoist for the elevator were advanced from time to time as erection progressed. However, the highest point reached by the elevator was several panels below the top of the steelwork and ladders were used to cover the additional distance.

Erection of Cables and Suspenders

Equipment for Unloading and Handling Materials.—The delivery of materials not only for the permanent structure, but for the temporary facilities necessary for the erection of the permanent structure, presented quite an unusual problem. The greater part of all the material required eventual delivery to the anchorages several hundred feet back from the waterfront and approximately 200 ft above the river level. Railroad connections to the site, particularly on the New Jersey side, were not to be found, and topographical features on both sides of the river made it rather impractical to consider trucking from the nearest railroad terminals. The contractor, therefore, adopted the only logical method—delivery of materials by water, on barges and car-floats, to or adjacent to the bridge towers, supplemented by cableway transportation to the anchorages.

On the New Jersey side provisions for mooring barges and car-floats were in place as installed by the contractor for the towers and floor steel. Two Chicago booms, 125 ft. in length, attached to the north and south faces of the tower, were used for unloading all material. They were operated by electrically driven hoisting engines placed on the shore with manila rope block-and-fall swing lines, and were capable of placing material on a temporary dock constructed north of the north pier and on a cantilever platform erected on the south side of the tower at Panel Point 0. The south boom was also long enough to reach a material track constructed along the shore line. A light, hand-operated derrick car on this track made it possible to use a level area of ground south of the bridge for storage purposes. This area was particularly necessary for the storage of the footbridge deck.

From the base of the tower, materials were hauled up the face of the Palisades, by means of a cableway with a capacity of 15 tons. A $2\frac{3}{4}$-in. track rope was attached to Tower Column *D*1, at the northwest corner, just below Panel Point 1, and passed over a 65-ft structural steel tower erected on top of the Palisades north of the approach cut. The track rope was anchored with a $1\frac{1}{8}$-in. wire line block and falls by means of which the sag of the

rope could be adjusted as desired. A material track was then used to transport the material from the cableway to a 10-ton crane operating on a runway erected across the approach cut over the west half of the anchorage floor steel. At the east end of the floor steel, over the areas not reached by the crane, materials were handled by a guy derrick.

On the New York side, the contractor elected to make use of a dock, south of the tower, constructed by the contractor for the anchorage. A stiff-leg derrick was erected at the north end of this dock and was capable of unloading all material, placing it either on a loading platform south of the south tower pier, or on a material track extending past the east face of the south tower pier. A caterpillar crane was used for distributing materials for storage on areas not reached by the stiff-leg derrick. As was the case on the New Jersey side of the river, a cableway was used to transport materials from the track at the tower to a second track on the ground between the buttresses of the anchorage. One end of the cableway was attached to Tower Column *A*8, at the southeast corner, just below Panel Point 2, and the other end was attached by a block-and-fall to a special structural steel anchor embedded in the concrete of the anchorage block. A stiff-leg derrick erected on top of the anchorage block on the center line of the bridge served to hoist the material from the material track and distribute it where necessary on the anchorage.

Concurrently with the installation of these facilities, the contractor for the towers proceeded with the erection of the grillage girders and frames for attaching the footbridge ropes, after which he hoisted the footbridge ropes and erected the cable saddles and the steel construction towers.

Grillage Girders.—Each grillage is made up of four longitudinal girders 31 ft long and 5 ft $1\frac{3}{4}$ in. deep, resting directly on top of the lateral plates forming the top plane of the tower bracing. A 6-in. slab carries the load of the saddle from the rollers to the longitudinal girders. As delivered in the field for erection, the entire grillage for each saddle was assembled in two sections which, when erected, were joined on the transverse center line of the tower. The girders of the sections were joined by a butt-stiffener splice. The two sections of slab had no splice.

Little difficulty was encountered in setting the grillages so that the slab would come in a true plane. In one case, because of a slight warp in the grillage section, it was necessary to omit the riveting of one butt-splice temporarily until after the load of the cable had remedied the condition.

Footbridge Ropes.—The construction of parallel wire bridge cables, with the possible exception of those for very short spans, always requires the use of working scaffolds or footbridges. For economy, it has become the practice of contractors to make use of the suspender ropes temporarily, to support these footbridges. The ropes are thus ordinarily manufactured in sufficient lengths to reach from anchorage to anchorage. After the main cables have been spun, these ropes are then cut into the proper lengths to form the suspenders. In principle, this was the procedure on the George Washington Bridge. However, due to the long span, it was practically impossible to

handle the rope in one piece of sufficient length to reach from anchorage to anchorage. They were cut in three lengths, one for each side span and the third for the main span. Two groups of nine ropes, $2\frac{7}{8}$ in. in diameter, spaced 22 ft apart, were used to support each footbridge. These groups were arranged with seven ropes in a central hexagonal cluster with one on either side.

The contractor elected to cut the ropes to length and socket them at the shop, and to make the necessary field adjustments for correct sag by varying the relation of the ends of the ropes to their attachments on the towers. Inasmuch as the space available for making these adjustments was relatively small, it was essential that the ropes be cut as accurately as possible to the correct lengths. To secure the required degree of precision, it was necessary to remove all slack inherent in the manufacture of wire rope, to accomplish which each length of rope was pre-stressed under a pull of 200 000 lb at the manufacturing plant, for a period of at least 12 hours. After being measured the rope was cut into the desired lengths, socketed with bail sockets, and placed on reels for shipment.

At the bridge site, the reels of side-span ropes were mounted at the tower bases and the ends were pulled up the hillside to the anchorage where they were attached, by means of structural and cast-steel links, to anchors embedded in the concrete above and on either side of the main cable eye-bars. (See Fig. 7.)

The reels containing the center-span ropes were mounted four at a time on a barge, and after the exposed ends had been connected by pennant lines to their corresponding side-span ropes at the base of the New York tower, the barge was towed as directly as possible to the New Jersey side, the ropes being allowed to unreel and sink to the bottom of the river as shown in Fig. 8.

At the New Jersey tower base the main-span and side-span ropes were connected by pennant lines. The ropes were then in position for hoisting to the tops of the towers, an operation that was accomplished one rope at a time. A short box-girder projecting over the side of the tower at the top, was used to hoist the rope to a point just below the tower top. At this point (the limit of hoisting with the "cat-head" equipment), the load was transferred to the derrick on the top of the tower, which hoisted the ropes the remaining distance and placed them in position in the structural steel frames riveted to the towers on either side of the main cable saddles. During the operation of hoisting the ropes, and while laying them on the river bed, it was necessary to stop all river traffic.

With all the thirty-six ropes in position, the individual lengths in each span were adjusted to hang at predetermined sags, such that with the addition of the footbridge decking and other miscellaneous material, the footbridges would be parallel to the unloaded position of the main cables and about 4 ft below them.

Of necessity, all adjustments were made at the tops of the towers, the attachments of the ropes at the anchorage being such as to preclude the pos-

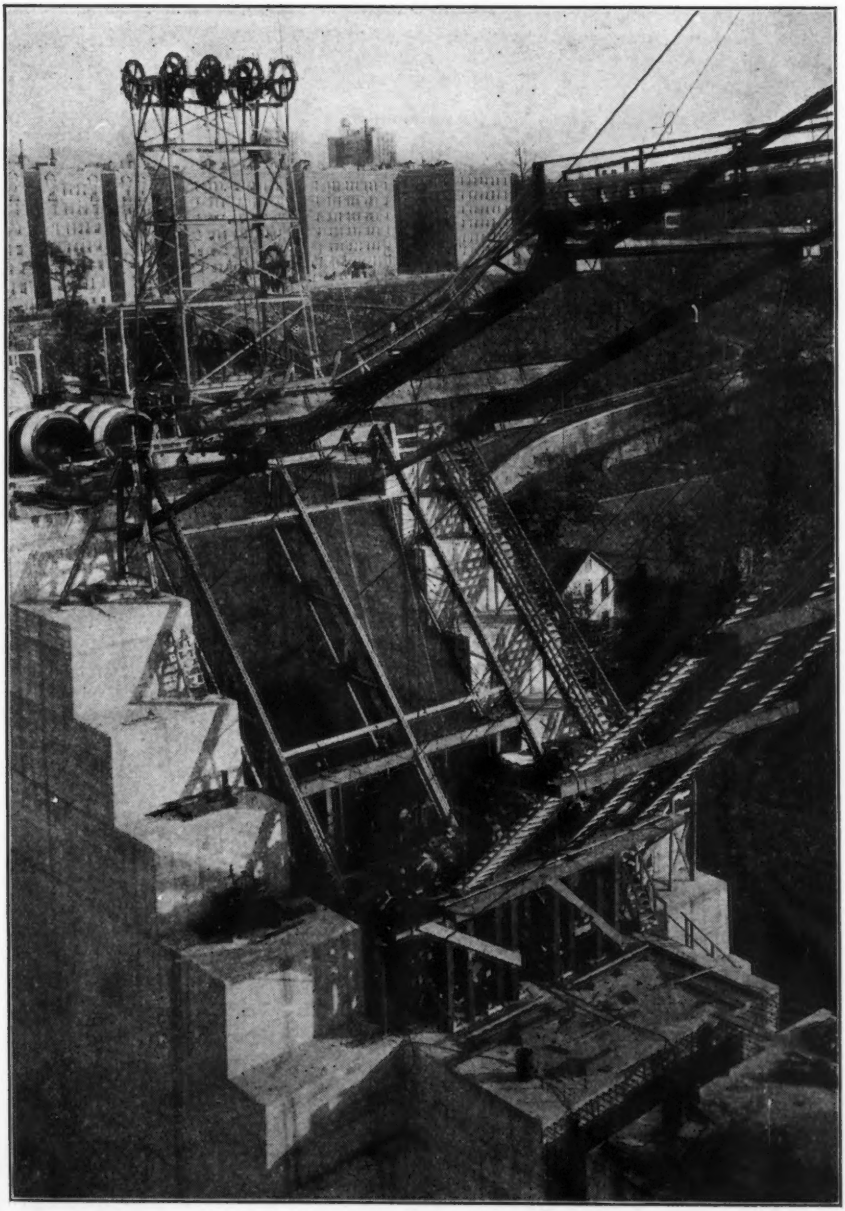

FIG. 7.—SPINNING EQUIPMENT, NEW YORK ANCHORAGE.

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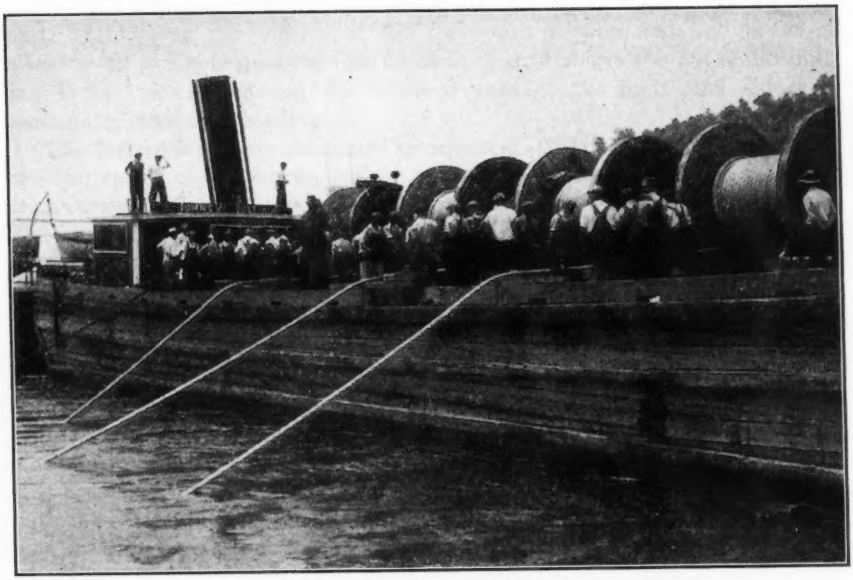

FIG. 8.—LAYING FOOTBRIDGE ROPES IN HUDSON RIVER.

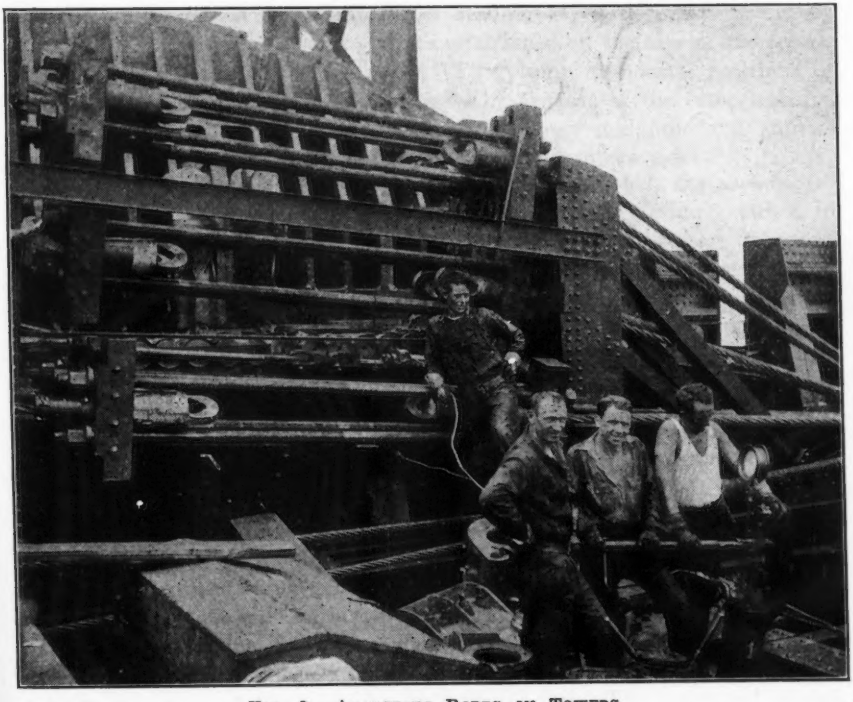

FIG. 9.—ADJUSTING ROPES IN TOWERS.

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sibility of adjustments at that point. The side-span ropes were adjusted first, by blocking the sockets of the main-span section and pulling in or slacking off the side-span rope as desired. Fig. 9 shows the hydraulic pulling jacks inserted between the socketed ends of the main and side-span sections of rope to be adjusted.

The desired sags were calculated in advance. Charts were prepared giving the full range of sag corresponding to all changes in span length resulting from tower deflection, and for all reasonable variations in temperature. The chart for the side spans, in addition, gave for each sag the station and the elevation at which a transit should be set up to establish a line of sight parallel to the chord and tangent to the curve having the desired sag.

Plumb-bobs could not be used to measure the deflections in the towers as is customary, because not one of the tower columns was vertical in its normal position and, therefore, could not be used as a well in which to suspend a plumb-bob. The method adopted required the use of transits set up on the piers, sighting on horizontal targets attached to the tops of the towers. The transits for the purpose were equipped with right-angle eyepieces, permitting true zenith observations. These observations were always made in pairs with the horizontal axis of the transit parallel to the center line of the bridge. The instrument was rotated 180° between the first and second of each pair of observations to eliminate instrumental errors. The targets, constructed at the bridge site, were graduated with a pointed stadia design and were artificially illuminated to permit direct readings to within 0.05 ft. They were set with a known relation to points established on the tops of the towers prior to hoisting the footbridge ropes. The plumb or normal positions of the towers had been determined previously by the use of the same transits, observations being made during the hours between midnight and sunrise when the steelwork was most nearly at uniform temperature.

After the side-span ropes had been completely adjusted, the sockets for these ropes were blocked and the main-span ropes were similarly pulled in or slacked off by jacking on one or both towers, as was necessary to group the ropes at the correct sag. The first rope in each group was set tangent to the line of sight of a level instrument set up in one of the towers at the elevation determined from the chart for the observed temperature and tower deflections. A target was also set at the same elevation on the opposite tower for use as a foresight to eliminate all corrections due to curvature of the earth surface, errors in adjustment of instrument, etc. Telephones at the instrument platform, tower tops, and bases, with a special tie-line across the river, permitted intercommunication of information relative to deflections, instructions for adjusting the ropes, etc., between the instrumentmen and the workmen.

As has been stated, one rope only in each group was adjusted by instrument, the remaining ropes being grouped to as nearly the correct relation to the first as could be observed with the aid of telescopes. Throughout the entire operation of adjusting footbridge ropes, cross-heads with threaded tie-rods were used to connect the side-span and main-span sections and were kept tight to carry the stress of the rope if a jack should fail. The structural frames

supporting the ropes were designed to permit separation of the individual ropes to provide clearance for these rods and also to carry only the unbalanced pull of the footbridge ropes; that is, the side-span ropes and the tower together resisted the pull of the main-span ropes. When the adjustments were completed, shims were left between the frame and the sockets of the rope sections of the main span.

Cable Saddles.—The anchorage saddles served not only to take care of the change in direction at the anchorages, but also as splay castings for the various strands composing the cables. Inasmuch as the center-to-center distance of the cables of each pair was considerably less at the saddles than at the eye-bar groups, it was necessary to set the saddles of each pair in planes inclined to each other as well as longitudinally, in order that their bases might be normal to the cable reactions. The saddles were also set with their axes at an angle to each other rather than parallel, as would be the case if there was no change in horizontal direction of the cables at this point. The saddles, which weighed approximately 21 tons each, rested upon segmental rollers which, in turn, rested upon 4-in. steel slabs. (See Fig. 10.)

The concrete of the anchorage structures was finished to the approximate planes for setting the slabs, allowance being made for about 2 in. of grout. Wooden templets designed to give the desired inclination of the slabs were used to set them to approximate position. Survey points were established on the center line between cables and on reference lines just riverward of the slabs. Computed offsets to these lines, together with elevations at each corner, were used in the final setting before grouting. The segmental rollers were then assembled on the slabs and blocked temporarily until the saddles were erected.

The anchorage saddles were then erected, slings being devised to suspend them at the desired angle. Wire-rope bridles and turnbuckles served to set and maintain the saddles in position until the first few sets of strands were in place and the resulting friction was sufficient to maintain them. This position was about $2\frac{1}{2}$ in. shoreward of normal, in order to allow for stretch in the back-stay due to the dead load of the structure.

After the footbridge ropes were erected, points were established on the grillages at the tower tops for roller and saddle erection. A careful procedure was followed to insure the correct alignment of the cable saddles. A point on the center line of the cable was established at the shore end of each grillage. A transit mounted on a trivet was then set up over these points and, with the points on the far tower as foresights, the center line of cable was established at the river end of each grillage.

This method of establishing the center line of cables on the grillages served to correct for any possible skew in the tower, which, however, was very slight. The rollers, spaced by end bars, were then placed in position on the slab and carefully retained in proper relation to allow for the motion of the saddle relative to the grillage.

Each cable saddle is made up of four separate castings, the heaviest weighing approximately 55 tons. The section nearest the shore span was

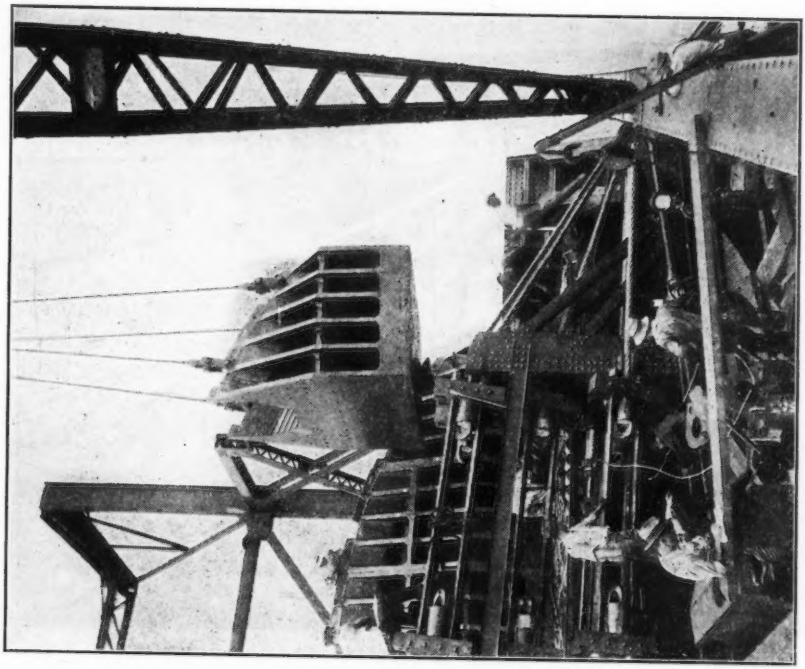

FIG. 11.—ERECTING TOWER SADDLE CASTINGS.

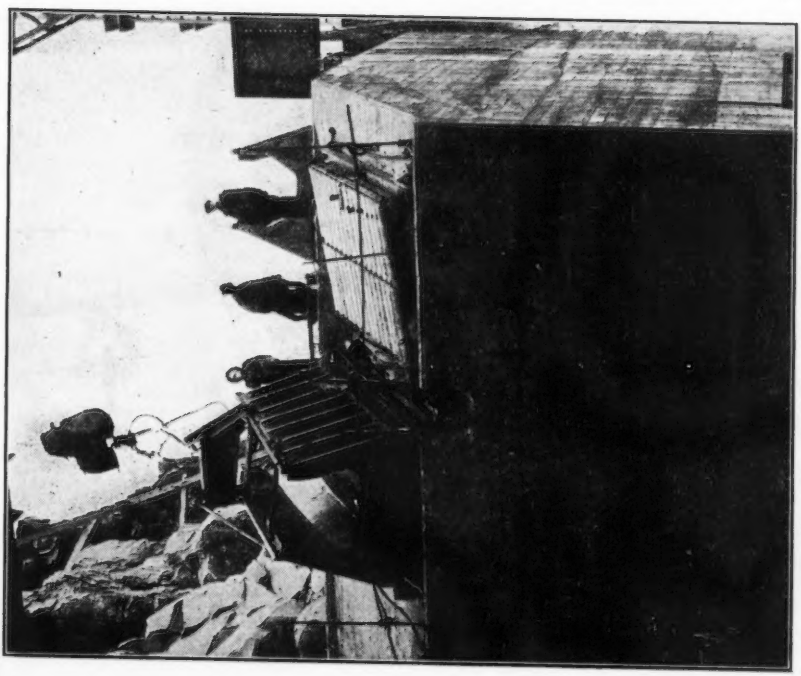

FIG. 10.—SETTING ANCHORAGE SADDLES.

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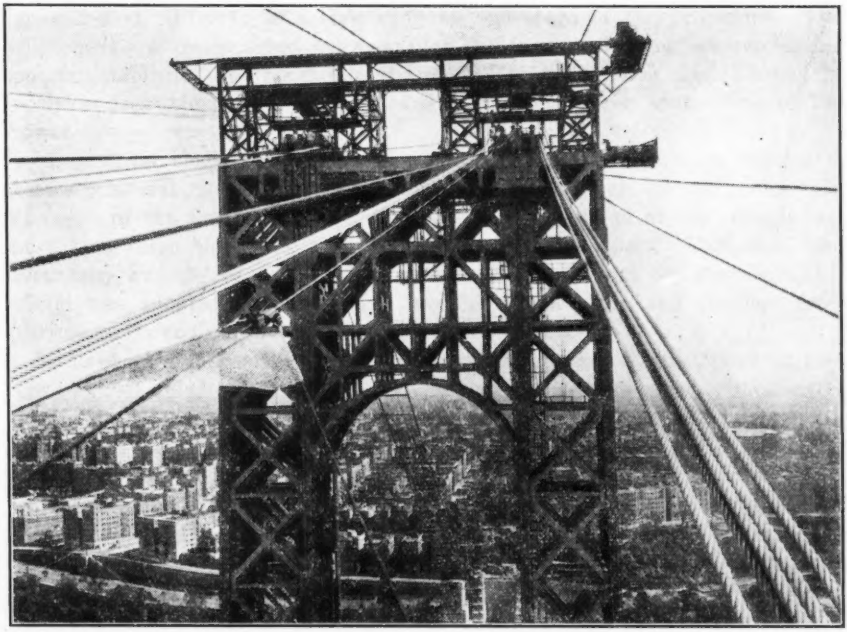

FIG. 12.—TOWER FALSEWORK.

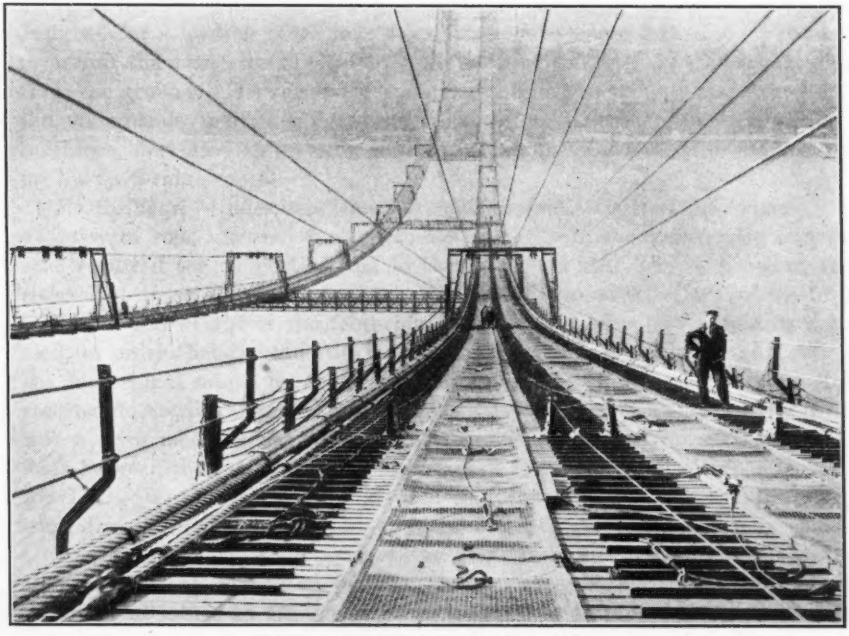

FIG. 13.—FOOTBRIDGES IN MAIN SPAN.

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erected first. Fig. 11 is a view showing one stage of the procedure. The alignment was maintained by means of the transit sighting on center-line points established on the saddle sections. The saddle was then blocked in position approximately 2 ft shoreward of the transverse center line of the tower.

Segmental blocks were provided at both tower and anchorage saddles to restrain lateral motion of the strands in the upper half of the cables. In the case of the tower saddles, however, the final position of the strands was such that these blocks did not provide the desired support. Therefore, lead antimony was melted and poured between the blocks and the strands. This metal was adopted due to its low melting temperature and freedom from shrinkage on cooling.

Falsework at Top of Tower.—The erection equipment at the tops of the towers was assembled after the cable saddle castings were in place. It consisted primarily of a steel superstructure carrying a crane runway 51 ft above the top of the main steelwork as shown in Fig. 12. Two track girders extended the full width of the tower (transverse to the bridge axis). Each was supported by four pairs of braced columns, one pair on either side of the main cable saddles. Cross-bracing between the column rows stiffened the structure in a direction parallel to the center line of the bridge. Lateral stiffness was supplied to the track girders by trusses with horizontal bracing in the planes of the top and bottom flanges. Within the areas occupied by the cable saddles the runway was designed to withstand a load of 130 tons; and, over the area, between the two pairs of cables, a load of 15 tons. Cantilever brackets designed for a loading of 15 tons were attached to either end of the runways to permit the cranes to run out over the ends of the towers to hoist material from the ground. The falsework was also designed to support platforms over the main cable saddles which were required to support electrically driven machines, first used in erecting the footbridge decking and, later, in reeling up the footbridge ropes.

Footbridges.—Under conditions normally found in suspension bridges, the quantity of rope required for suspenders in the permanent structure exceeds that required for supporting the footbridges. On this bridge, however, the reverse of these conditions existed, so that it was essential to reduce to a minimum the weight of the footbridge decking and other miscellaneous dead load, in order that the quantity of excess rope required solely for supporting the footbridges might be reduced to a minimum. Therefore, the contractor resorted to steel framing partly covered with wood decking and partly with woven wire mesh as shown in Figs. 13 and 14. A central walkway, 5 ft wide between the cables and two walkways, 3 ft 6 in. wide at either side, were provided. Woven wire mesh was used to cover the areas between the walkways directly under the positions to be occupied by the completed main cables.

For the main-span footbridges the steel framing was fabricated at the shop in sections consisting of 8-in. channel cross-beams spaced 12 ft back to back and joined by six longitudinal trusses made up of a bent-rod Warren web system welded to chord angles (see Section *B-B,* Fig 14). The trusses

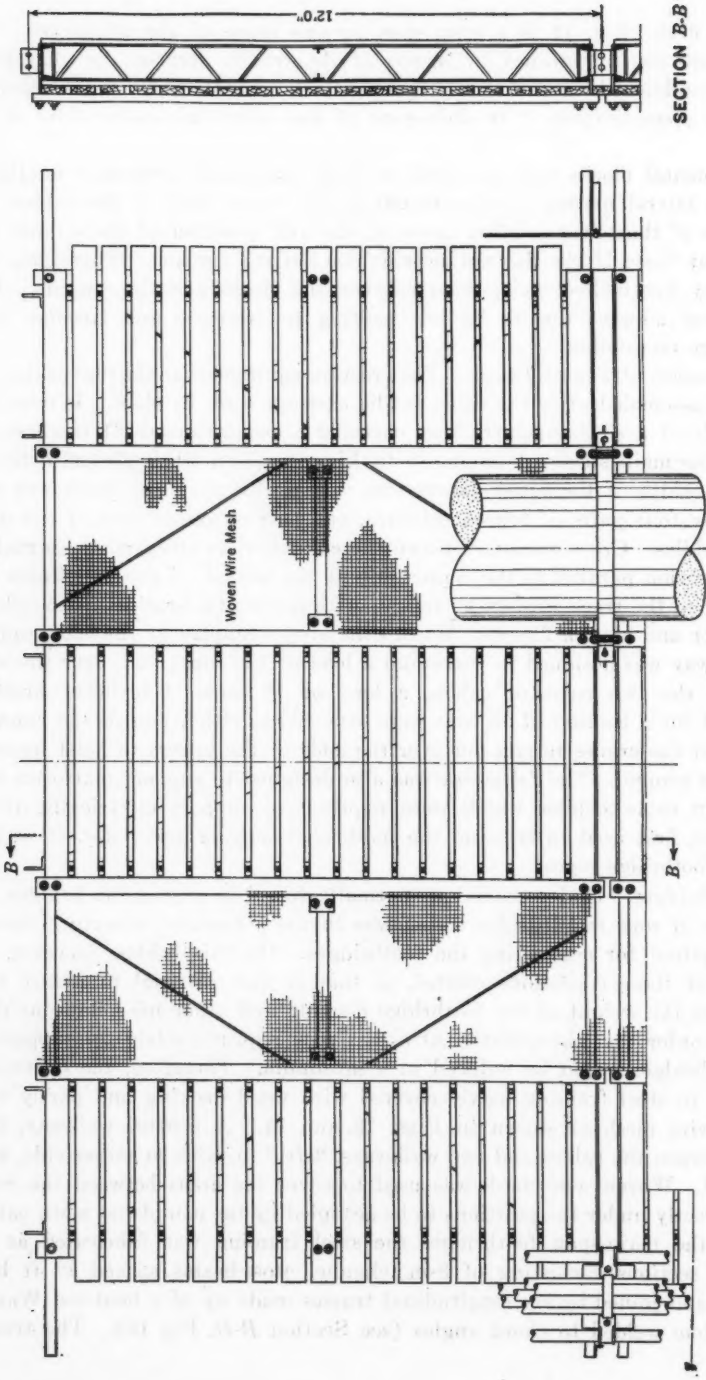

FIG. 14.—PLAN OF MAIN SPAN FOOTBRIDGE.

were spaced to support the edges of the three walkways and the entire unit braced by diagonal tie-rods. The sections were bolted to the underside of the footbridge ropes by means of bent straps over the ropes. (See Fig. 15.)

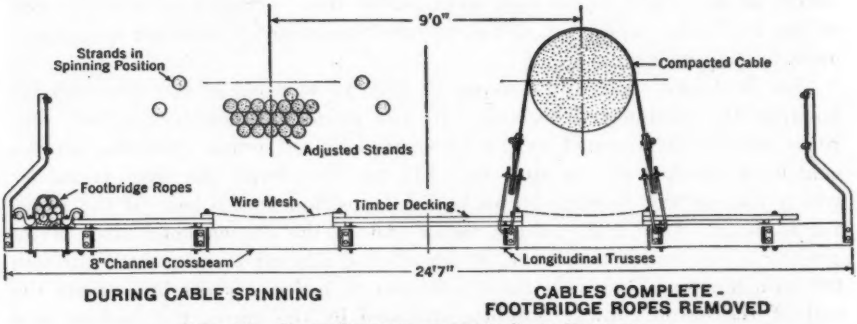

FIG. 15.—CROSS-SECTION OF MAIN SPAN FOOTBRIDGE.

The side-span footbridges were designed as a series of steps and landings because of the steepness of the slope. As in the main span, each footbridge provided for three walkways located to give access to the cables. The walkways were formed by units consisting of pairs of trusses designed so that the top chord and diagonals of the trusses could be used to support two flights of steps and two intervening landings. These units were supported on 10-in. pipe cross-beams spaced 30 ft apart and suspended from the footbridge rope groups by means of turnbuckle suspenders. The explanation for the suspension of the sections in the side span is that the contractor had intended originally to build the cables by the method of successive erection and had designed the anchorage for the footbridge ropes accordingly. Under the successive program the footbridges would have had to be lowered to clear the loaded position of the first pair of cables. To meet this condition, the contractor had designed the sections to be suspended from the footbridge ropes rather than support them on top of the ropes, as is done customarily. The plans for the equipment had progressed so far that, at the time decision was reached to construct all four cables simultaneously, the contractor preferred to utilize the original method of supporting the footbridges from the ropes. In the main span, however, the footbridge sections were brought into contact with the ropes instead of being suspended as in the side spans. Chemical fire extinguishers were installed in cages suspended at intervals beneath the footbridges to guard against the destruction of the footbridges by fire, as happened in the construction of the Williamsburg Bridge at New York, N. Y. As an added precaution, all timber was painted with fire retardant paint and sheet metal fire stops were placed under the decking at approximately 600-ft spacing.

After the falsework at the tops of the towers was completed and after the footbridge ropes had been adjusted, the footbridge decking was erected. This operation could not be carried out in the usual simple manner, by sliding the sections out from the towers over those previously placed, because

of the method adopted for supporting the decking. Different methods of erection also had to be used in the main and in the side spans.

For the main span, the footbridge sections were hoisted to the tops of the towers in skip cars, where they were loaded into carriages designed to run on the footbridge ropes in which they were transported to position for attachment to the rope groups.

The L-shaped steel frame shown in Fig. 16 was one of the skip cars for hoisting the sections of decking. It was guided by counterweighted skid ropes while being hoisted to the tower tops by the crane operating on the cantilever bracket of the runway. Fig 16 also shows the steel frame by which the car was brought to an inclined position at the base of the tower for loading. Steel links at the upper end of the car engaged steel plates riveted to the top of the tower, which allowed the car to be rotated through 90° to a horizontal position for attachment to pennant ropes hung from the end of the crane runway. When unloaded by the crane, the decking was transported over the cable saddles, deposited on the tower tops near the center line, and thence relayed, one section at a time, to hand-cars operating on the temporary platform shown in Fig. 17, which was attached to the river face of the tower below the level of the footbridges. The carriages riding on the rope groups were loaded by hoisting several sections into the opening provided for the purpose.

Footbridge sections were erected from the middle of the span toward the towers, with four carriages such as that shown in Fig. 18, two operating from each tower. Each carriage consisted of a rigid steel frame pin-connected at each corner to a two-wheeled truck. The wheels were made of maplewood restrained between steel plates, with the rims shaped so as to conform to the grouping of the footbridge ropes. Wooden blocks enclosing the group of ropes were mounted on arms in front, and in back, of the wheels to hold the ropes in their correct relation with each other. The blocks were divided on their horizontal axis to facilitate easy removal and re-attachment. This was a necessary precaution to permit clearing the erected sections of decking.

An under-slung steel cage was hung from the frame by means of a single pin at either side, an arrangement which permitted the cage to hang vertically regardless of the varying inclination of the frame corresponding to the slope of the ropes. The cage was designed to form two longitudinal working platforms, one above the other on each side, and two transverse platforms over each other across each end. Easy access to the various platforms was provided by ladders in each corner. Hand winches on the lower transverse working platforms of the cage were used to hoist the footbridge sections into the cage. These winches were arranged so that they could also be driven by pneumatic drill machines to speed up the operation of hoisting the sections.

The carriage was then shifted along the ropes, propelled by an endless wire rope (later used as a hauling rope) passing around idler sheaves at either anchorage and around an electrically driven spool mounted on the temporary platform above the main cable saddles. When in position for erection

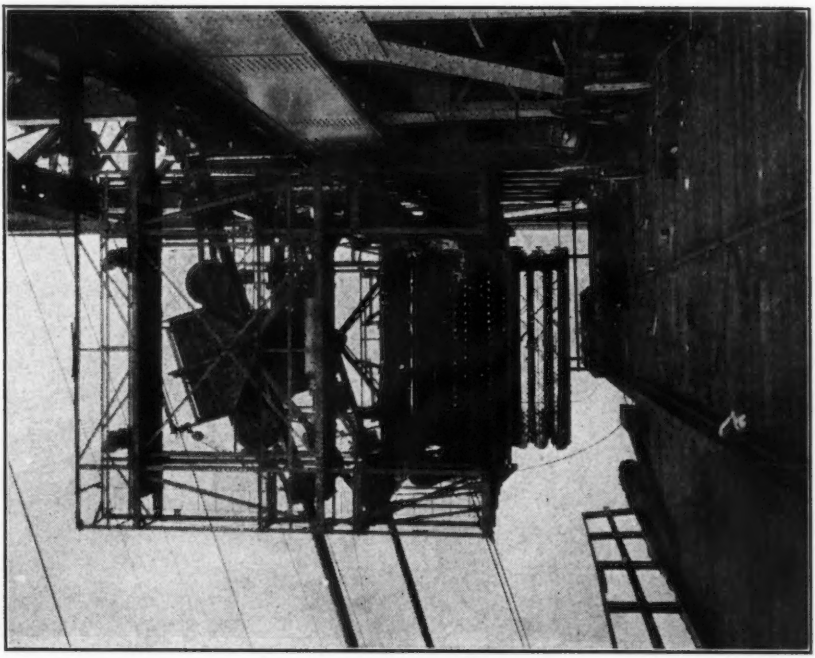

FIG. 17.—LOADING MAIN SPAN CARRIAGE.

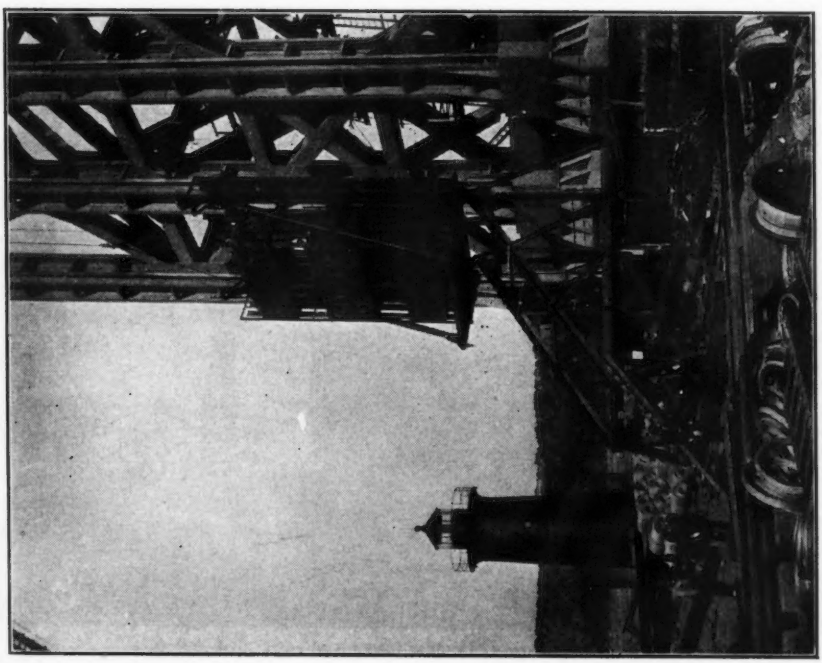

FIG. 16.—SKIP FOR HOISTING SECTIONS.

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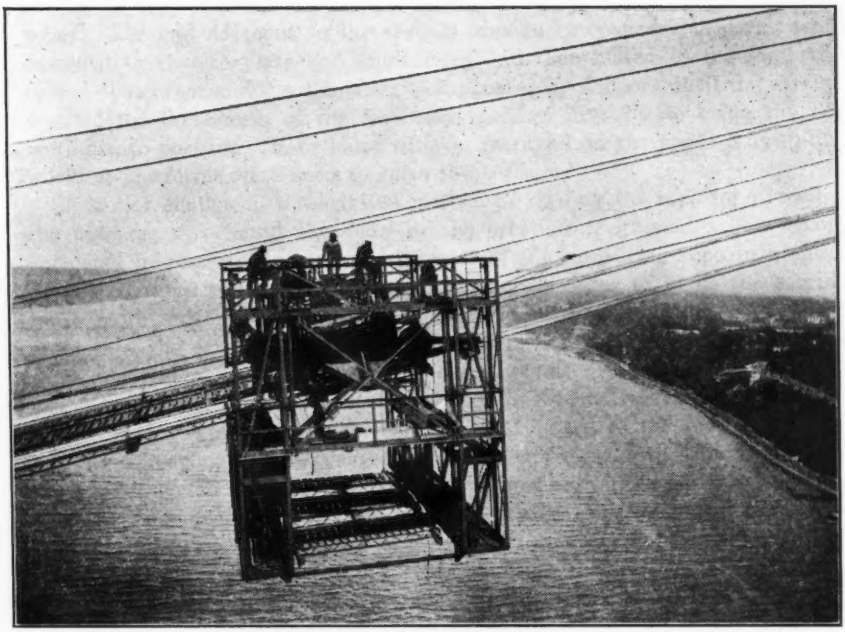

FIG. 18.—MAIN SPAN CARRIAGE.

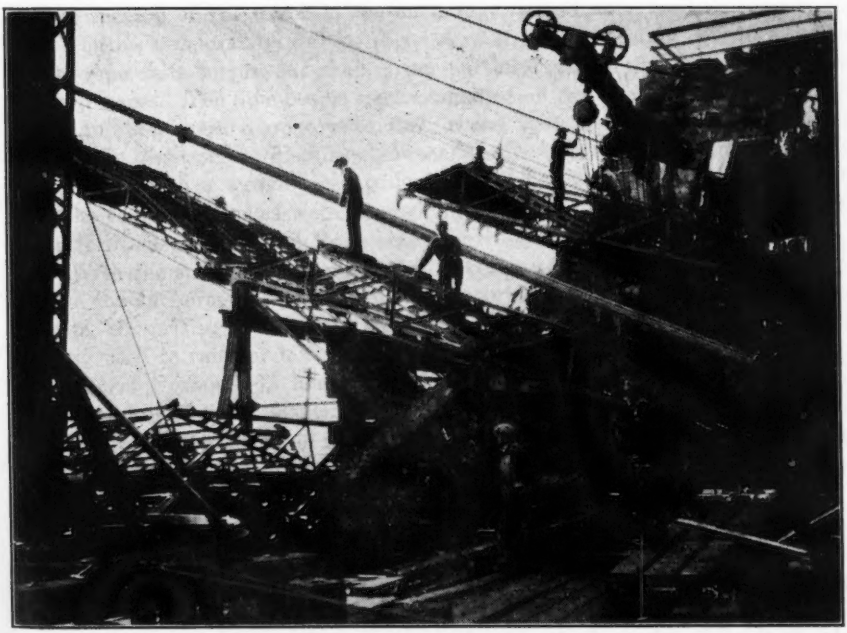

FIG. 19.—ERECTING SIDE SPAN DECKING.

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the top section of the pile was raised into contact with the under side of the ropes. The end adjacent to the section previously erected was permanently fastened to it with plate and angle clips and then bolted to the footbridge ropes. The other end was temporarily supported by hangers until the carriage was shifted the length of the footbridge section in order to bring the next section into position. Four hand winches mounted on the upper longitudinal working platforms were used to raise the sections.

A minor change in the original method of rigging the rope for propelling the carriage was found necessary as the erection progressed. This change consisted in restraining the opposite part of the propelling rope in a sheave on the carriage to limit its variation in sag, in order to make the carriage move more uniformly toward the middle or flatter part of the span. Aside from this, the carriages functioned very well, and the erection of the main-span footbridges was completed in eight working days. Five steel truss cross-bridges connected the north and south footbridges in the main span. These were hoisted directly from barges and bolted to the footbridge ropes.

The side-span decking was erected simultaneously with that of the main span in order to balance the pull of the ropes on the towers. The units forming this decking were also erected (from the anchorage toward the towers) by means of carriages operating on two ropes stretched above, and approximately parallel to, the footbridge ropes. Before these units could be erected, however, the pipe cross-beams forming their support had to be placed and suspended from the footbridge ropes. The ends of the pipes were equipped with steel castings that engaged a pair of 2-in. wire ropes stretched from the top of the tower to the anchorage, which were used to support the pipe beams temporarily. These ropes were anchored through counterweighted wire-rope falls to prevent them from participating in the load of the footbridge ropes. The pipe beams were assembled on the 2-in. ropes at the anchorage and were pulled up toward the towers, spacer ropes serving to fix and maintain the correct distance between beams. They were then clamped to the ropes with a wedge locking device and the suspenders connecting them to the footbridge ropes were erected, the workmen making use of a hanging platform suspended from the erection carriage.

With the suspenders in place the decking was transported by the carriages from the anchorage to position in the footbridges as shown in Fig. 19. Three units, comprising a complete panel, were carried on each trip, chain blocks being used to raise or lower them as necessary.

Storm System.—It has been the practice to stiffen footbridges against excessive deflections resulting from severe winds or storms. This is usually done by stretching wire ropes between the tower bases. These ropes are arched upward nearly to meet the footbridges at mid-span by means of vertical hangers attached to the footbridge ropes. Additional stiffness adjacent to the towers is sometimes obtained by the use of diagonal guy ropes from one or more points on the footbridge ropes to the towers at approximately the floor level. Due to the unprecedented span of the George Washington Bridge, the contractor was unwilling to rely upon this system of storm ropes. In an effort to find a more adequate arrangement, a scale model of the footbridges

was constructed upon which experiments were made with various systems of ropes. The one adopted as being the most effective in dampening both vertical and lateral deflections contained the salient features of the systems commonly used, with certain additions.

All the ropes were of high-strength steel, $\frac{1}{2}$ in. in diameter, except as indicated in Fig. 20. The arched ropes, two for each footbridge, were attached

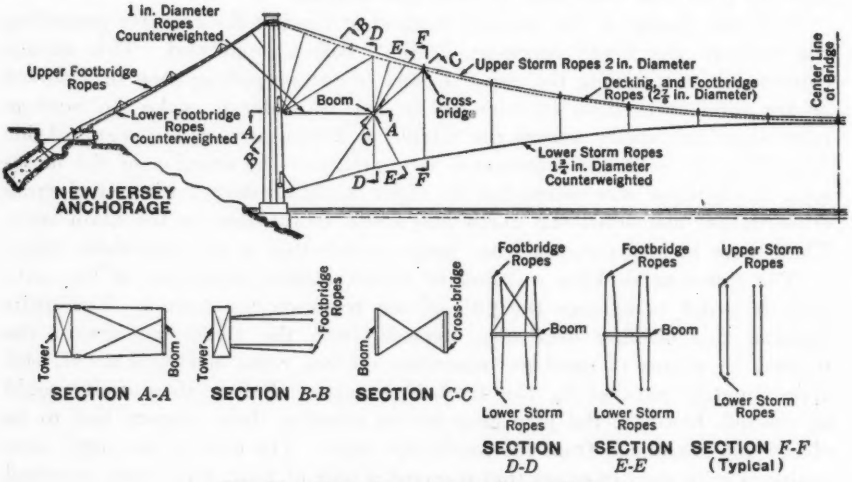

FIG. 20.—CROSS-SECTIONS OF STORM SYSTEM.

at one end by means of counterweighted wire-rope falls. The hanger ropes were attached to ropes suspended above the footbridge ropes and the footbridges were connected to the hanger ropes.

Latticed angle struts, about 42 in. square and 140 ft long, were suspended beneath the footbridges at either end of the main span, 310 ft from the towers and approximately 300 ft above the water. Four sets of diagonal ropes connected each strut to three points on the footbridge ropes and to two points on the lower storm ropes. The struts were also braced laterally with ropes to the footbridge ropes and to the towers. Those connected to the tower passed through sheaves at the towers and back to attachments on the struts, the sheaves, in turn, being fastened to the tower by means of counterweighted wire-rope falls. The counterweights at the various connections to the towers were used to maintain relatively constant stresses in the storm system for any variations in temperature or loading of the footbridges. In actual use the storm system proved very effective, no deflections being noticeable in the footbridges, except during a few periods of severe winds.

At various points on the footbridges and storm system and also on the tower tops electric lights were installed for the protection of aerial navigation.

Method of Spinning Parallel Wire Cables.—The basic principles of spinning parallel wire cables have become well known by reason of the large number of suspension bridges built in recent years. They were used more than seventy-five years ago by the late John A. Roebling, M. Am. Soc. C. E.,

in constructing the Niagara Suspension Bridge and, again, by his son in building the well-known Brooklyn Bridge. Essentially the same principles, with certain refinements, have been used ever since.

For the benefit of those who may be unfamiliar with the construction of parallel wire cables, the following brief description is given.

As its name implies, a parallel wire cable is simply a bundle of parallel wires attached at either end of the bridge to steelwork built into structures, known as anchorages. The wires making up the cable are divided, for purposes of construction, into several units or groups called "strands," which lose their identity in the completed cable except at either anchorage where the various strands splay out to separate connections to eye-bar chains and

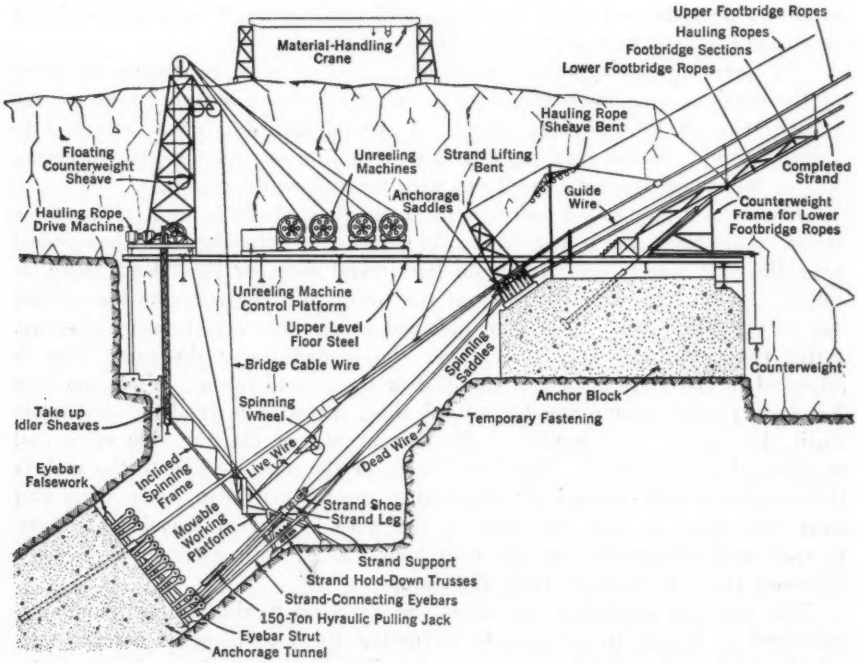

FIG. 21.—SECTION AT NEW JERSEY ANCHORAGE.

girders forming the anchorage steelwork. The wire in any one strand is continuous and is looped around a horseshoe-shaped steel casting or "strand shoe" at either end, which is inserted between a pair of eye-bars and pin-connected to them. The entire arrangement is shown diagrammatically in Fig. 21.

The strands are usually constructed in individual supports or saddles outside the main cable saddles with the strand shoes temporarily connected to the eye-bars by means of "strand legs," which are steel castings constructed so that one face of the strand shoe is exposed to permit the wire to be looped around it. A strand is spun by first taking the exposed end of a reel of wire

mounted at the anchorage and attaching it to any convenient point in front of the strand shoe. The wire is laid around the strand shoe and a loop of wire is formed, one part leading to the strand shoe and the other to the reel. This loop is then progressively lengthened by pulling wire from the reel, until it reaches the other anchorage where it is placed around the strand shoe at that point. The wire leading to the reel is then placed around the near strand shoe again, forming a second loop which is similarly lengthened and placed around the far strand shoe. This operation, called "spinning," is repeated until the desired number of loops or wires have been placed in the strand. The wire leading to the reel is then cut and spliced to the end previously fastened in front of the strand shoe, thus making the wire continuous. The loops of wire are pulled from anchorage to anchorage by placing them around a deep flanged wheel known as a "spinning wheel" which is attached to an endless tramway or "hauling rope.

To speed up the spinning operation, a reel of wire is usually mounted at the opposite anchorage and wire is pulled out for a second strand on the return trips of the spinning wheel. A second spinning wheel attached to the other part of the hauling rope and traveling in the opposite direction to the first wheel is used to pull out wire for two more strands.

The wires are adjusted individually to the desired sag in each of the three spans. This operation is as follows: As soon as the spinning wheel has passed over the first tower, the wire from the strand shoe, or the "dead" wire, is pulled through the strand saddles at the tower until it hangs at the correct sag in the side span. It is then clamped at the tower to prevent slipping. After the spinning wheel passes over the second tower the same wire is adjusted in the main span and clamped at the second tower. When the loop has been placed around the far strand shoe, the wire is rendered around it until the "dead" wire hangs at the correct sag in the far side span and is clamped at that point. The "live" wire, or the wire leading to the reel, is then similarly and successively adjusted in the far side span, main span, and near side span, so that all slack in the wire is carried ahead from span to span and ultimately into the next loop. After both wires of a loop are adjusted, they are removed from the clamps.

The sags for adjusting the wires are fixed by "guide"-wires which are measured to length in advance by adjusting them to hang at certain sags established by instrument in a manner similar to that used in adjusting the footbridge ropes. After a small group of wires are in place in each strand, the "guide"-wires are removed, and the remaining wires are adjusted to the group. The lengths of "guide"-wires are changed according to the computed variations in the lengths of the strands and are used at the start of each set of strands without having to be set every time by instrument.

When a set of four strands has been spun, the wires in each strand are bunched together with a nominal seizing. The strand is then transferred from the strand saddles to final position in the cable saddles, the strand shoes at the same time being removed from the strand legs and permanently connected to the eye-bars (see Fig. 15).

After all the strands have been completed and placed in their final position in the main saddles, the cables are compacted or squeezed in the spans between the saddles. The squeezing breaks up the circular strands and fills in the voids between them to form a single compact circular cable as shown in Fig. 15.

The number of strands to be used in constructing a cable is generally fixed so that they can be built up in layers to form a hexagonal cross-section prior to squeezing. Thus 7, 19, 37, 61, etc., may be used. The required cross-sectional area of the cable combined with the limiting size of strand determines which multiple to use. As has been previously mentioned, sixty-one strands, each containing 434 No. 6 wires were used to make up each of the four cables for the George Washington Bridge.

Adjustment of Guide-Wires.—The guide-wires were set just prior to the completion of the erection of cable-spinning equipment. Four wires for each cable were pulled over the footbridges by hand. The galvanizing had been omitted from these wires to make them easily distinguishable from the cable wire. As has already been mentioned, these wires had to be adjusted to the correct sags as established by instrument.

The sags are usually measured in both main and side spans. However, in this case, due to the relatively short side spans as compared to the main span, it was decided to follow a procedure that would require instrument observations in the main span only. The wire to be measured was mounted on 1-in. steel rollers, spaced 1 ft apart on a steel strap laid in the center groove in each tower saddle. When so placed, the wire immediately adjusted itself to balance the tension on both sides of the saddle, in such a way that fixing the sag in the main span would automatically fix the sags in the side spans. The total length of guide-wire could thus be measured very accurately. The wire was then pulled in or slacked off at one anchorage the amount necessary to secure the desired sag in the main span, allowance being made for variation in temperature above or below the normal, which was 50° F. With this method of adjustment, tower deflections could be neglected since they had practically no effect on the sag because of the free movement of the wire on the rollers. Observations were made by means of a level instrument set up on the tower, sighting on an inverted target rod lowered through the footbridge decking at mid-span. Only one wire of each set was adjusted by instrument, the remaining three wires being made to correspond to the first.

Improvements in Cable-Spinning Equipment.—Several new features were devised for spinning the cables of the George Washington Bridge. These functioned very well and made it possible to increase the rate of spinning cable wire beyond anything previously accomplished.

An improved method of mounting the reels of cable wire at the anchorages was introduced. It had been the practice simply to mount the reels in a rack, relying upon the pull of the spinning wheel to rotate the reel. Manually operated brakes served to slow down or to stop the reel when necessary. The inertia of the reels, particularly when filled with wire, would cause considerable surging and whipping of the live wire along the footbridges whenever the speed of the spinning wheel varied.

To avoid these difficulties, the reels were mounted on machines that rotated them in a vertical plane at the speed required to unreel the wire as pulled out by the spinning wheel. A specially designed reel was used, consisting simply of a steel plate cylinder, 6 ft in diameter, with bent channel flanges riveted to either end. The reels each had a capacity of about 8 tons of cable wire, a continuous length of about 30 miles. They were slipped on and off an overhanging cylindrical member of the machine with the aid of hand-cars. Six telescoping arms extending out from the cylinder as shown in Fig. 22 served to raise the reels clear of the hand-cars and lock them in place as shown in Fig. 23.

Eight unreeling machines, two for each cable, were required at each anchorage. The driving power had to be capable of rotating the machine at varying speeds up to approximately 75 rpm, and also capable of changing from one limit to the other of this range within a few seconds. Hydraulic power was adopted as being most suitable for these conditions. It consisted of an electrically driven hydraulic pump, the output of which could be varied in amount and also changed in direction by means of a manually operated regulator. The pump was connected with high-pressure pipes to a hydraulic motor geared to the unreeling machine. One 100-hp electric motor was used to drive a set of four pumps serving the four unreeling machines for a pair of cables at each anchorage. Only two of the four machines, of course, were ever in operation at the same time. On the New York side the machines were set up on top of the anchorage block, behind the eye-bar pits. Those on the New Jersey side were set up on the floor steel directly over the eye-bar pits. (See Fig. 21.)

As an indication to the operator of the speed of the unreeling machine in relation to the speed of the spinning wheel, the wire from the reel was run through a compensating sheave tower where momentary variations between rates of unreeling and pulling out were absorbed. This also maintained practically uniform tension in the "live" wire, eliminating all tendency for it to whip along the footbridges. Two deflecting sheaves were mounted in line at the top of a structural steel tower, as shown in Figs. 7 and 21. The wire coming from the reel was led over one deflecting sheave, down under a floating sheave, and then up over the other deflecting sheave. The operator of the reel machine varied its speed to maintain the floating sheave as nearly as possible at the mid-point of its range of travel.

The wire of one reel was spliced to that of the next with a sleeve coupling pressed into place with a three-jaw hydraulic press. This splice was identical to that used in the shop for the manufactured lengths of wire.

Another quite important departure from customary practice was the manner in which the spinning wheel was attached to the hauling ropes which required a new type of hauling rope support. The spinning wheel was mounted on the side of the rigid, triangular, structural steel frame shown in Fig. 24, the two upper corners of which were supported by pin-connected steel-plate links from two steel plates parallel to, and directly under, the hauling rope and attached to the rope by two steel straps. These straps

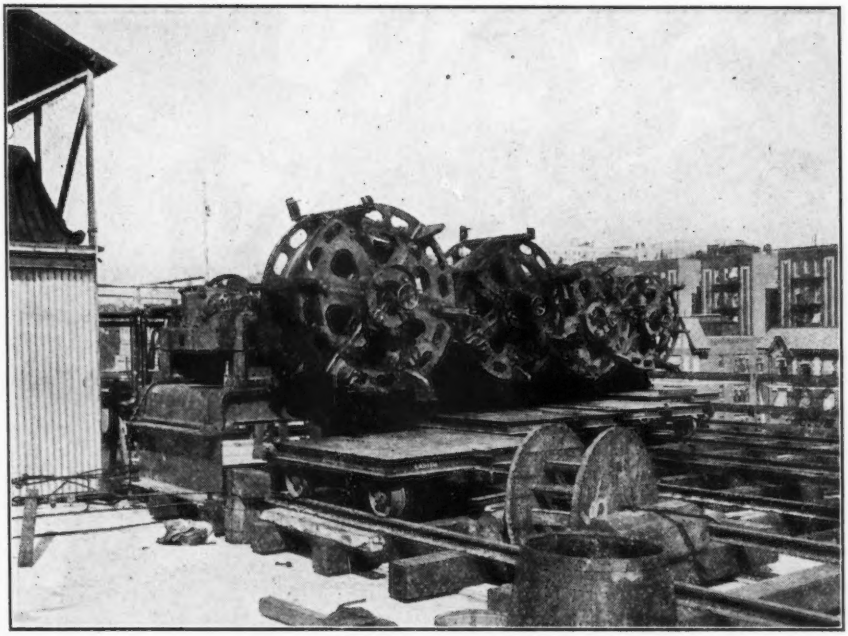

FIG. 22.—VIEW OF UNREELING MACHINE.

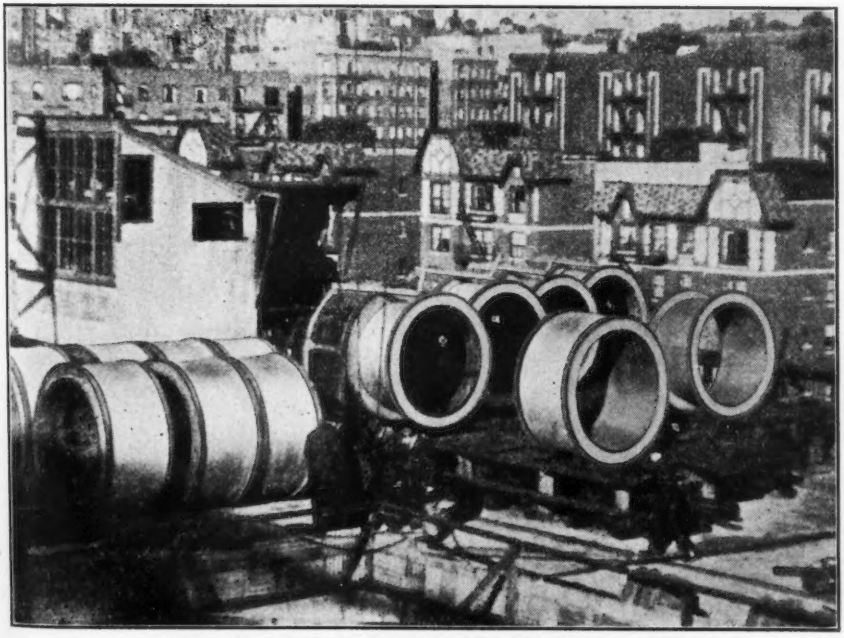

FIG. 23.—REELS MOUNTED ON UNREELING MACHINES.

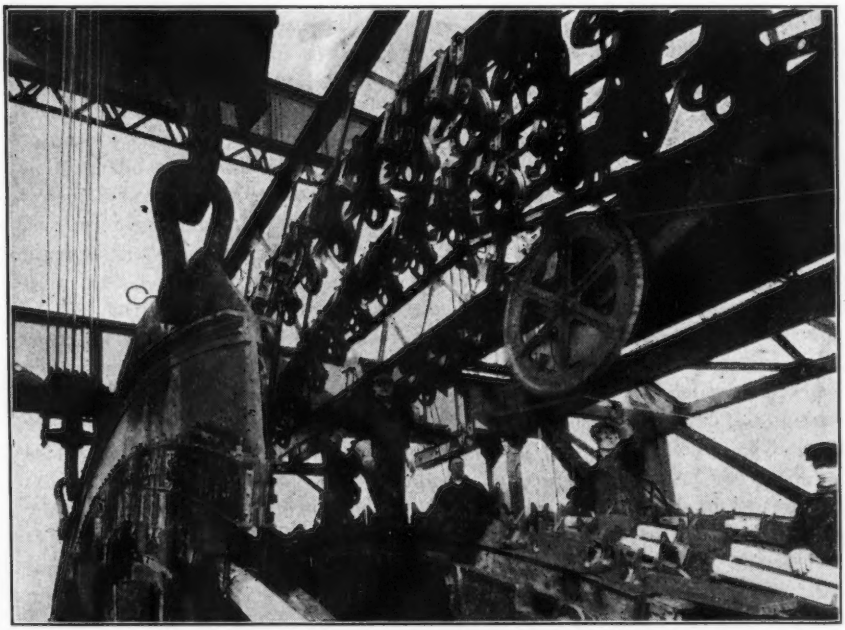

FIG. 24.—BALANCE BEAM AT TOWER (ATTACHED TO STRAND BUT NOT LIFTED).

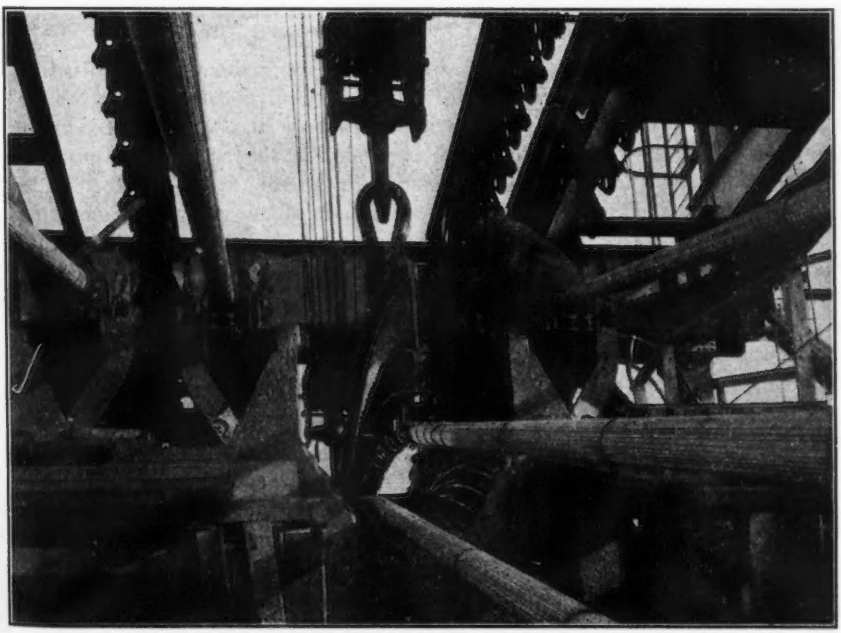

FIG. 25.—BALANCE BEAM PLACING STRAND IN SADDLE.

were bent around a bronze collar clamped to the rope. The collar permitted the rope to rotate within the straps and eliminated any tendency of the lay to tighten or loosen at these points. The pull of the hauling rope was transmitted to the spinning wheel by means of a third plate similarly attached to the rope directly over the wheel, and pin-connected to the frame. The pin-hole in the plate was slotted in a direction normal to the hauling rope in order to provide for movement between it and the frame while the rope is passing over the sheaves. The intervals of rope between the straps were filled with collars of the same diameter as the straps, and tapered collars were placed at each end, to create smoother operation over the sheaves.

An entirely new type of sheave was required for supporting the hauling ropes, because the attachment to the spinning-wheel frame was made under the rope rather than on one side as had been the former practice. The sheave designed to meet this condition consisted of two independent wheels mounted opposite to, and at an angle with, each other, such that the distance between their flanges was a minimum at the top. The clearance between the flanges was fixed just so as to allow the straps supporting the spinning-wheel frame to pass. Pins attached to the axles of the wheels and projecting through spiral slots cut in the caps clamping the axles to a cast-steel yoke, served to adjust this clearance quickly. The flanges of the wheels were machined to form a groove centered directly over the gap between them, the lower part of which was of a diameter such as to fit the hauling rope and the upper part of a larger diameter to fit the straps and collars of the spinning-wheel attachment. A series of these sheaves were placed in an arc at the towers and over the anchorage saddles, to take care of the change in direction of the hauling ropes at these points. At the steel bents, which were spaced at about 225-ft intervals along the footbridges, idler sheaves were also mounted above, and in line with, the support sheaves. The tension of the hauling ropes was kept sufficient to maintain a slight upward reaction at these points, except during the passage of the wheel, thus reducing to a minimum the wear on the hauling rope resulting from the partial support of the double-wheel sheaves.

The hauling ropes passed around idler sheaves at the New York anchorage. The driving equipment and the arrangement for maintaining tension in the rope were located at the New Jersey anchorage where the ropes passed around driving drums and around adjustable idler sheaves directly below the drums. Adjustment was secured by wire line falls pulling down on the idler sheaves.

The ropes were driven by means of 100-hp reversible electric motors through "Philadelphia" reduction gear units. At first, the maximum speed was set at approximately 620 lin ft per min. It was later increased to 675 lin ft per min, or 70% greater than had been attained previously on similar work.

Movable working platforms supported over the anchorage eye-bars just behind the strand shoes (shown in Fig. 7) were also an important feature. As the sets of strands were completed, these platforms were moved up on a

pair of inclined trusses erected on either side of the eye-bars. The hauling ropes and the live wires from the compensating towers passed around deflection sheaves mounted on these platforms, since it was necessary to bring the spinning wheels and the wire loops within easy reach of the workmen.

A dispatcher on the New Jersey side maintained complete control of the operation of the spinning wheels by means of an electric signaling system. Push-buttons connected to signal bells at each anchorage, were placed on the working platforms at the eye-bars, at each tower, and at every hauling rope bent along the footbridges. Each of these buttons was also connected to an annunciator board in the dispatcher's house. The workmen at each anchorage would signal when they had their loop of wire in place around the spinning wheel and after receiving the signal from both anchorages the dispatcher in turn, would signal the hauling rope operator by buzzer to start the wheels. If, for any reason, it was necessary to stop the wheels at some point along the footbridges, the men could so signal the operator with the push-button nearest that point. The dispatcher knew immediately at what point the wheels had been stopped and by telephone could determine the reason. When the trouble had been corrected, a starting signal over the same push-button notified the dispatcher to signal the operator to start the wheels. This arrangement eliminated much lost time in locating the source of any trouble and also the likelihood of accidents resulting from some one signaling to start the wheels from some point other than that at which they had been stopped.

As previously mentioned, the wires were adjusted individually to hang at the correct sag in each of the three spans. This is usually done by attaching to the wire beyond the supports through which the wire is to be rendered, a lineman's wire-gripping tool commonly known as a "come-along." The "come-along" is connected, in turn, to a light manila rope falls with which workmen stationed at these points pull up or slack off on the wire as signaled by an adjuster stationed at the mid-point of the span in which the wire is being adjusted. Ordinarily, this method is entirely adequate but in the case of the George Washington Bridge it had several disadvantages. In order to adjust the wires it was necessary to pull them up in tension to approximately 600 lb., a greater force than could easily be applied single-handed without makng the workman travel excessive distances in adding more parts to the falls. Furthermore, the length of the main span was too great to permit visual signaling from mid-span to the towers. Therefore, mechanical units at the "come-alongs," controlled by the adjusters in the spans, were adopted. The units, one set for each cable, were installed on either side of both towers and at each anchorage. Each unit consisted of an endless rope that passed around two sheaves mounted along the foot-bridge and thence down to an electrically driven drum placed beneath. Pennant lines attached to the rope and to the "come-alongs" thus performed the same function as the block and falls. The electric motor driving the unit was reversible and was controlled either by the man at the "come-along" or by the adjuster in the span.

A system of standing electric light signals was also adopted to indicate when the wires were ready to be adjusted and when the adjustment had been completed. For instance, when a dead wire was adjusted in the near side span and clamped, the men at the tower would turn on a pair of lights, one at the tower and the other at the middle of the main span. When the spinning wheel had passed over the far tower and the men had the "come-along" in position on the wire, they would turn on a pair of lights similarly located. Both lights burning at mid-span was a signal to the adjuster to proceed with the adjustment of the wire. When completed to his satisfaction, he would turn out both sets of lights which served as a signal to the men at the far tower to clamp the wire and remove the "come-along." Side-span adjustments were made in an identical manner. One set of lights in each span served for the wires in the pair of strands pulled out by one spinning wheel and an identical set served for the wires of the other pair of strands.

As an aid to the adjusters in the main span, men were stationed at the quarter-points to signal to the adjuster the positions of the wire at those points. This precaution was necessary in order to adjust, properly, such wires as would not hang symmetrically due to slight non-uniformity of weight, wind, conditions, etc. Later, it was found that results just as satisfactory could be obtained more quickly by steadying the wire at the quarter-points at the correct relation to the group, thus reducing the effective span length for the adjustment to about one-half.

Extremely accurate adjustments were obtained by this method. When a strand was completed it was customary to remove all banding throughout its length, thus allowing all wires of the group to hang freely for final inspection of adjustment. It was not at all unusual to find all the wires in the main span hanging within a range of 1 ft. Wires hanging decidedly above or below the group were cut and lengthened or shortened as necessary and spliced with threaded sleeve couplings.

Seizing, Placing, and Adjusting the Strands.—The strands were compacted into their circular form approximately $4\frac{1}{2}$ in. in diameter by means of specially designed tongs so constructed that they would lock automatically when a certain diameter had been reached. They were easily adjusted to accommodate slight variations in strand diameter. The strands were compacted at approximately 5-ft intervals and were seized with cotton bands having metal clips. Another type of seizing, consisting of several wraps of cloth tape treated with an adhesive compound, was adopted for the later strands. It is the opinion of the writers that neither of these types of seizing was entirely satisfactory as it is believed that they did not break during the cable-squeezing operation to allow the wires to become fully re-adjusted so as to fill the voids. This may be partly the cause of the difficulty encountered in squeezing the cables, as mentioned later. This belief was strengthened upon observing the action of these bands while squeezing a model section of cable to check the calculated cable diameter.

The parts of the strand that would be placed in the tower and anchorage saddles were seized with a series of wire wrappings of several turns each

in the spaces between the strand saddles. At the anchorage the two halves of each strand were drawn together 24 ft from the strand shoe and seized for a length of 12 in. with $\frac{1}{8}$-in. stranded wire.

Balance beams such as that shown in Fig. 24 were used to lift the strands from the spinning position in the strand saddles to permanent position in the cable saddles. At the towers they were built in the form of girders with flanges curved so as to be approximately parallel to the curve of the strands passing over the tower. The strand to be lifted was attached to the bottom flange by a series of $\frac{3}{8}$-in. wire rope grommets, made up by hand from a single length of wire strand. These grommets were placed under the strand, and their ends were looped over small cast shoes attached to the lower end of adjusting screws on opposite sides of the balance beam. During the operation of placing the grommets the balance beam was supported above the strand by wooden blocks resting on the strand saddles so that the grommets could be adjusted to nearly uniform tension.

The strand was then lifted by two sets of 12-part falls hung from either end of the crane, and was shifted laterally and lowered into place in the tower saddle as shown in Fig. 25. This operation was carried on at one tower at a time to reduce the hazard of a strand tearing loose from the balance beam. While being lifted at the tower, the strand was similarly lifted at the adjacent anchorage saddle by a shorter balance beam attached to a 60-ton hydraulic pulling jack which was hung from a carriage rolling on a pair of I-beams. This arrangement is shown in Fig. 26. The beams were pivot-connected at each end to the sides of a steel bent erected over the saddles. The carriage support permitted the strand to be shifted laterally from spinning position to the final position, and the pivoting of the beams eliminated eccentric pull on the carriage during the operation of pulling back the strand to make the permanent connection at the strand shoe.

Before lowering the strand into position in the saddle, steel fillers shaped to fit around the under-half of the strand were inserted between the parts of each grommet. The fillers were a little thicker than the grommets. Strips of thin sheet metal were placed above and below the fillers to facilitate the withdrawal of both grommets and fillers from under the strand. The strips were then pulled out if possible; if not, they were cut off and left in place.

The first strand placed in the main saddles for each pair of cables was adjusted to hang at the correct sags established by instrument. In the side spans this was done according to the method described for adjusting the footbridge ropes and in the main span as described for adjusting the guide-wires. The first strand of the second cable of each pair was adjusted to hang level with the strand in the first cable which had been adjusted by instrument. The remaining strands of all cables were then adjusted to their proper relation with the strands already in place.

As first placed in the saddles, the strands were set to hang approximately 1 in. or 2 in. high in each of the spans, leaving the grommets under the strands at the anchorage saddles. Observations were made early the following morning when the strands were more nearly at uniform temperature, to determine the amounts by which the strands were too high. The strands

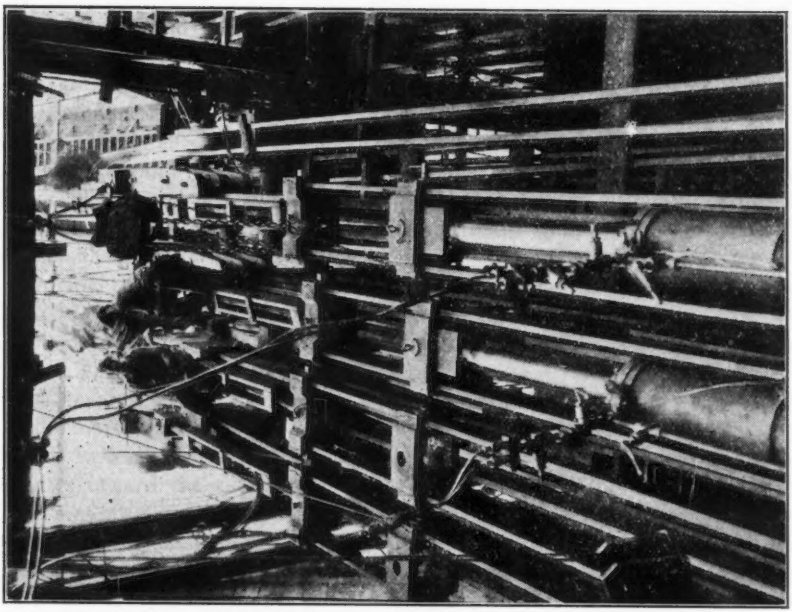

FIG. 27.—HYDRAULIC JACKS AT ANCHORAGE.

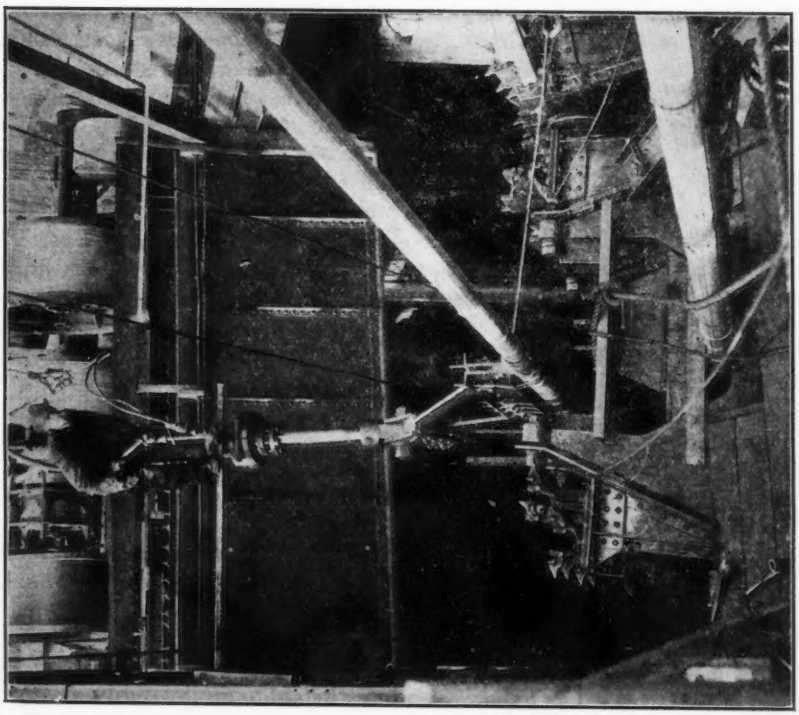

FIG. 26.—BALANCE BEAM AT ANCHORAGE.

were then rendered through the saddles during the day the amounts necessary to lower them to their correct positions. These amounts were calculated from the known sag ratios, allowances being made for the effect of removing the grommets at the anchorage saddles. The strands were easily rendered through the tower saddles by supporting them with chain falls from the bents that held the hauling ropes in the side spans, and slacking off at the strand shoe. They were always held clear of the anchorage saddles during this operation. The strand adjustments in the main span were usually divided between the two sides.

During the operations of spinning and adjusting, the tower saddles were blocked in such a position that the horizontal component of the adjusted strands in the main span equalled the horizontal component of the adjusted strands in the side span at an assumed normal temperature. Under such condition the actual tension in the side spans was approximately 13% greater than that at the main span side of the saddle, and a friction of 13% between the strands and the saddle casting was required to restrain the strands against slipping toward the side span.

Comparatively slight changes in temperature affected considerable changes in the relative tensions in the strand in the tower saddle as between the main span and side span. The sag in the main span, approximately 300 ft, was such that shortening of the wire due to decrease in temperature had little effect on the stress in the strand at the tower. However, the sags in the side spans were so slight (9 ft in one case and 11 ft in the other), that even slight changes in temperature made quite material changes in stress in the strands on the shoreward side of the tower. This condition required special consideration during the adjustment of the first few strands because of the relative stiffness of the footbridge system and the tower, which resisted the comparatively small unbalanced forces exerted on it by the strands.

At the time of adjusting the first strand in the side spans, the temperature had dropped considerably below that anticipated when the saddles were located on the tower tops. This resulted in unbalancing the side-span and main-span tensions sufficiently to overcome the friction, causing the strand to slip through the saddles into the side spans. This condition was met by jacking the saddles farther shoreward, after which the strands could be readily retained in adjusted position. The friction was also increased by removing the shop coat of paint that had been placed on the machined surface of the saddles. As additional insurance against further slipping, the contractor also attached clamps at the main-span side of the saddle to the two lower layers of strands. These clamps were blocked against the saddles.

The strands were adjusted at the anchorages by means of 150-ton hydraulic pulling jacks as indicated in Fig. 27. As shown in Fig. 21 the jack completed one leg of a two-part rope-and-link tackle that was attached at either end to the strand leg and passed around a split sheave placed on the pin at the lower ends of the strand-connecting eye-bars. Cross-heads and side bars by-passed the jack so that it could be extended for a second pull or removed entirely without dismantling all the tackle. Shims were kept inserted between

the lower cross-head and the adjustable lugs pinned to the side bars to prevent more than a limited movement of the strand in case the jack should fail.

In the past it has been the experience of engineers in constructing large parallel-wire cables, that as the upper layers of strands are set in place and adjusted, unequal temperature conditions caused by the sun shining on the top layers cause the strands to lengthen and force their way down between the strands in the layers below. Frequently, the strands become so wedged between those below that they will not resume their normal relation when temperatures again become uniform. To overcome this difficulty, strand retainers, consisting of wood-lined hexagonal steel frames, were attached to the adjusted strands at seven points in the main span and at the middle of the side spans. The members forming the upper parts of the frame were built in sections which were bolted in place as the upper layers of strands were spun and adjusted. The frames were attached to the two bottom layers of strands by means of thin steel straps looped around the strands and tightened with threaded rod extensions projecting through the bottom of the frame.

A peculiar condition resulted from the use of this method of attaching the retainers. A twist developed in the main-span cables, causing the group of strands to rotate in one direction in one-half the span and in the opposite direction in the other half. This condition was most pronounced in the outside cables, which rotated in opposite directions. The twisting finally became so severe as to hamper the strand adjustments seriously. It proved to be due to uneven temperature conditions which caused a longitudinal shifting of the strands in relation to each other. The "retainers" did not prevent the strands from shifting, but did prevent them from returning to their normal position. This was finally corrected by simply loosening the steel straps attaching the frames to the strands.

Unusual Construction Details Required at Anchorages.—Several features in the design of the steel for the anchorage required special consideration in the construction of the cables. For economy, the steelwork in the New Jersey anchorage was designed with a change in alignment between the strands and their connecting eye-bars, with the eye-bars converging toward a point considerably farther riverward than that of the strands. During the spinning operation this condition and the temporary positions of strands in spinning saddles outside the cable saddles tended to create excessive bending stresses in the embedded eye-bars at their points of projection from the concrete, since the strand-connecting eye-bars that were added and connected to the embedded bars at the time of spinning were allowed, during that operation, to follow the alignment of the wire strand. This latter condition also would have caused excessive stress in the eye-bars at the New York anchorage. The bending stresses in the projecting bars were practically eliminated by bolting the bars down by tie-rods and angle clips to a pair of I-beams erected beneath the bottom layer of eye-bars. The I-beams were rigidly connected to the falsework steel.

The change in alignment at the New Jersey anchorage caused greater difficulty, however, in the adjustment of the strands. It was necessary to

deflect the lower sets of strands and eye-bars to final position by pulling them downward at the strand shoes. The condition was a maximum at the lowest set of bars and decreased to zero at the middle set, above which point a gradually increasing upward deflection was required in the upper sets of bars.

The problem of adjusting and holding the lower sets of strands out of line was somewhat difficult. Studies were made as to the feasibility of placing anchor-bolts in the rock bottom of the eye-bar pits and also of excavating niches in opposite sides of the pits and inserting I-beams, or girders, to which the strands could be fastened. It was finally decided in the interest of economy and safety to use inverted truss arch ribs (see strand hold-down trusses, Fig. 21), hinged on the center line between cables and bearing against concrete pads built on the rock sides of the eye-bar pits. Four pairs of these ribs, located just below the heads of the eye-bars at the strand shoes, were used in each eye-bar pit. The six lower layers of strands were held in position with tie-rods connected between the arch ribs and the eye-bars.

In the finished cable the eye-bar heads are spaced by plate separators and the upward reactions of the lower layers of eye-bars are offset by the downward reactions of the upper layers.

The limited width of the eye-bar pits made it practically impossible to mount the strand shoes for spinning in the customary inclined plane above the eye-bar heads and still have room to spin four strands for each cable at one time. The strand leg, therefore, was designed to hold the strand shoe in a vertical plane in front of the eye-bar head. It consisted of two major parts: (1) A rectangular steel block of such dimensions as would fit between the pair of eye-bars with the upper end machined to bear against a pin placed through the eye-bar heads; and (2) a steel forging, the lower end of which fitted around the eye-bars in back of the heads, the upper end of which was provided with a swivel mounting for attaching the strand shoe. A removable side-plate bolted to the lower part of the forging prevented the eye-bars from spreading and slipping off the pin. Rectangular holes were cut through the forging and steel block for inserting a key which transmitted the pull of the strand to the block and, in turn, to the eye-bars. Pin-holes were placed in the ends of the key for attaching the adjusting tackle. Provisions were made for three different positions of the forging in relation to the block, corresponding to the computed variation in length of the strands. This was necessary to keep the strands as nearly as possible at a convenient height above the main-span footbridges.

Spinning Organization and Progress.—The erection of the cable-spinning equipment and the adjustment of the guide-wires was completed on October 18, 1929. Due to some minor adjustments required, actual spinning did not begin until October 21, when six trips of the spinning wheels were made on Cable *A* (the four cables were designated *A, B, C,* and *D,* from south to north) and three trips on Cable *C*. Spinning on Cable *B* and on Cable *D* was begun October 29 and November 11, respectively. The first set of four strands, on Cable *C,* was completed November 11.

The accompanying chart (Fig. 28), records the daily progress in spinning the wire for each of the four cables. The rapid increase in efficiency of operation is clearly indicated. The first set of four strands required an average of 22½ calendar days in spinning and 21½ calendar days in adjusting and setting up for the next set of strands, as compared to the later average of 10 days in spinning and 2 to 3 days in adjusting.

At first, the workmen were organized in four spinning crews, each of which adjusted the strands when they had been completed. This arrangement did not prove entirely satisfactory, principally due to the inexperience of the men. Too much time was lost in adjusting a set of strands and in setting up for the next set. Accordingly, late in January, 1930, the men were re-organized into three spinning crews and one strand-adjusting crew. The three spinning crews shifted in cycles from cable to cable. The strand-adjusting crew worked in rotation on all four cables. Progress was carefully supervised and regulated, the crews being ordered to work overtime if necessary to avoid any delay to another crew. A decided improvement in progress resulted from this change.

Spinning was completed on Cable *C* on July 28, 1930, followed by Cable *A* on July 30. The remaining cables, *B* and *D*, were completed August 1 and August 7, respectively.

The maximum quantity of wire spun in any one day was on May 7, 1930, when 220 trips of the spinning wheels were made, placing 880 wires totaling about 236 tons for three cables. On this day, two of the spinning gangs worked 11 hours, and the third, 10 hours. The maximum wire spun in one cable in one day was on March 19, when 81 trips of the wheels were made in 12 hours on Cable *A*, placing 324 wires, or about 87 tons of wire. The maximum rate of spinning wire in one cable for a period of 8 hours, or more, was 7.34 trips of the wheels per hour, or about 8 tons of wire per hour.

Squeezing the Cables.—Immediately after the sixty-first strand was adjusted, the cables were squeezed in the main and side spans to within about 20 ft of the saddles. The squeezing operation broke up the individual strands, filled the voids, and formed a compact circular cable, as shown in Fig. 15. This was done in two operations, a preliminary squeezing—which made the cable partly round by forcing the corner strands in—and the final squeezing.

The preliminary "squeezer" consisted of a hexagonal frame that surrounded the cable with hydraulic-jacks mounted radially at the corners (see Figs. 29 and 30). Each jack was fitted with a shoe that pressed against the cable, the inside face being shaped to fit approximately over the corner strand and the one adjacent on either side. The squeezer was flexibly suspended from a carriage which also held the hand-pumps for the hydraulic-jacks. This carriage operated directly on the cable, each end being supported by a two-wheeled truck that could be lowered or raised to adjust its position. The wheels were made of wood and were shaped to fit the top layer of strands. Near the towers the carriages were propelled with hoisting engines placed at the anchorages and on the flatter portions of the cables with hand falls. Squeezing in the main span began at the towers. A single

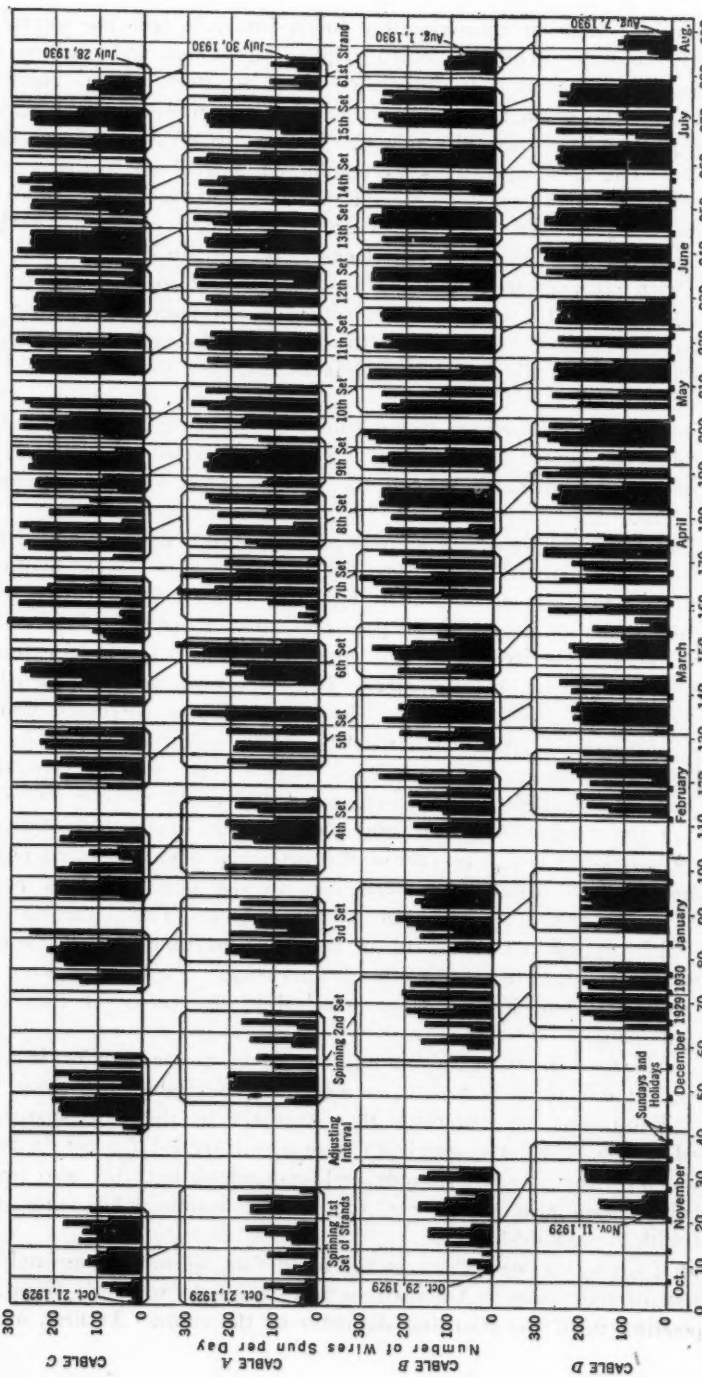

FIG. 28.—SPINNING CHART.

application was made at approximately 100-ft intervals, working toward the middle of the span. On the return trip to the tower, applications were made at approximately 12-ft intervals. A 2-in. steel seizing band was placed around the cable at each application. This was tightened with an auxiliary jack and fastened with a wedge lock set in a cast saddle as shown in Fig. 30. The same operations were followed in the side spans, working from the towers to the anchorages and return. The main-span cables had to be squeezed principally at night when the strands were more nearly at uniform temperature. The hydraulic-jacks were 3 in. in diameter and were operated with oil pressures up to about 5 000 lb per sq in. The pair of jacks on the horizontal axis was connected to one pump, the upper pair to a second, and the lower pair to a third pump. At each application the routine was first to apply the horizontal jacks and then the upper and lower jacks together.

Two machines were used for the preliminary squeezing beginning July 31 and finishing on August 14. During this time, the spinning equipment at each anchorage was dismantled, and the hauling rope bents were removed from the footbridges.

The final squeezers were very similar to the preliminary squeezers. Each consisted of a circular frame surrounding the cable with twelve hydraulic-jacks mounted radially, as shown in Fig. 31. The jacks were $5\frac{1}{8}$ in. in diameter and were operated with oil pressures up to about 6 000 lb per sq in. The inside faces of the jack shoes were concave and, with the jacks extended, formed a circle 36 in. in diameter. The squeezers were supported by carriages operating directly on the main cables. These carriages, two for each side of the river, spanned a pair of cables, being supported by two double-wheeled trucks running on each cable. The wheels were made of wood and were shaped to fit the compacted cable. An electric pump was mounted on the carriage for operating the squeezers, which were hung on out-riggers extending from one end of the carriage. Hand-operated pulling jacks were used to connect the squeezers to the out-riggers to permit adjusting their position in relation to the cables. The carriages were propelled with the hauling ropes, two of the drive machines for this purpose having been transferred to the New York anchorage. These ropes were also cut and passed around idler sheaves attached to the cables at the center of the main span.

The carriages were designed to pass over the tower saddles into the side spans, but due to their weight, it was felt inadvisable to attempt this transfer. Equipment for handling the squeezers in the side spans was improvised in the field. The carriages were also intended for use in transporting and erecting the cable bands and suspenders, but this also proved impractical because the wheels cut the seizing bands unless they were protected with curved steel plates.

The final squeezing was begun at the middle of the main span and progressed toward the towers. Applications were made at intervals from 2 to 4 ft, depending upon the resulting diameter of the cable. At first, all the

FIG. 29.—PRELIMINARY SQUEEZER.

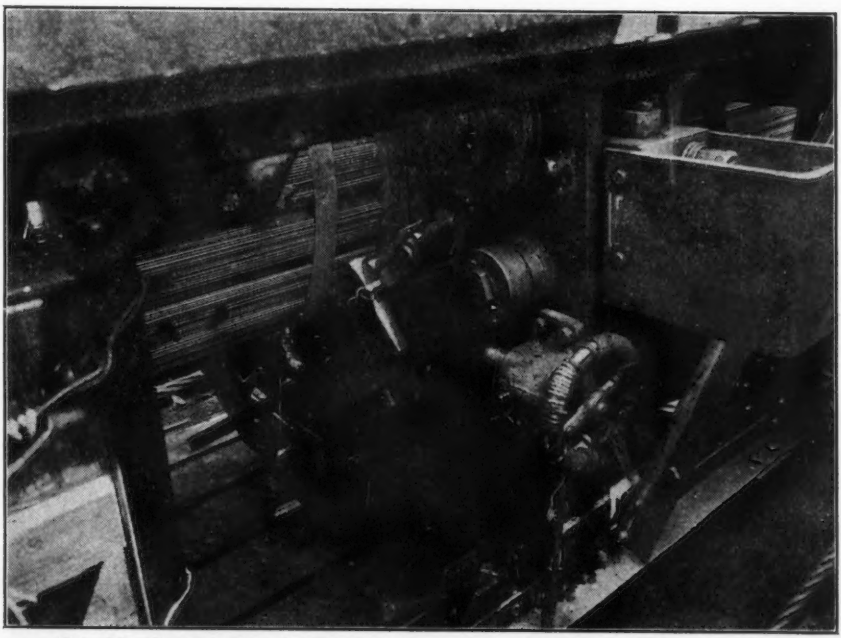

FIG. 30.—PRELIMINARY SQUEEZER; MECHANISM FOR APPLYING SEIZING BANDS.

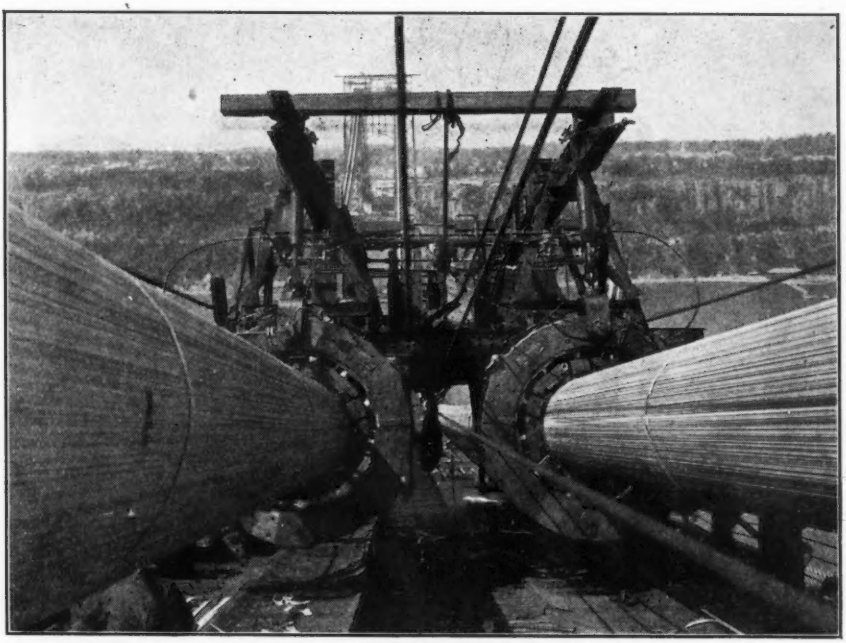

FIG. 31.—FINAL SQUEEZER IN MAIN SPAN.

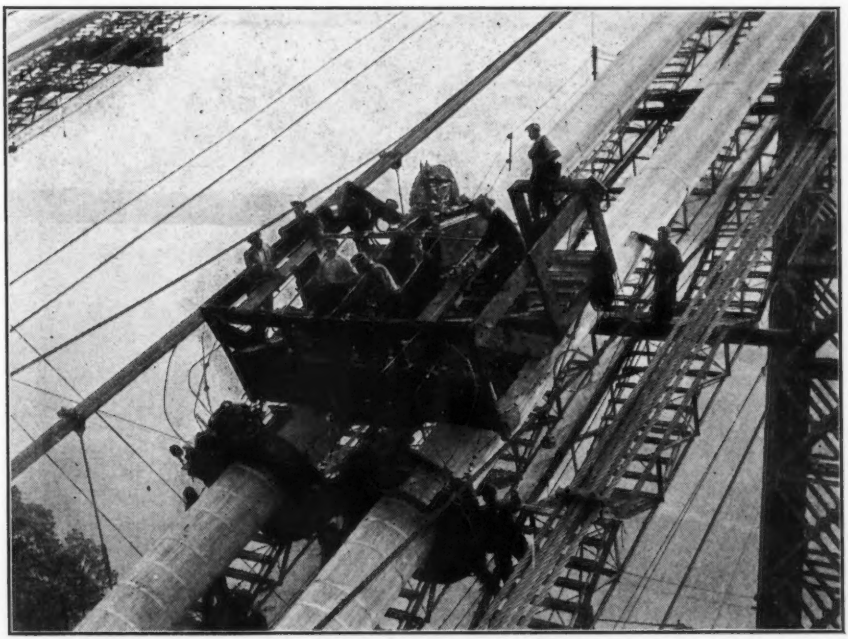

FIG. 32.—SQUEEZER IN SIDE SPAN.

jacks were connected as a unit, but this did not give satisfactory results, because the final cross-section of the cable was found to be elliptical with the horizontal axis a maximum. The connections were then re-arranged so that the three jacks on either side worked together as one unit and the three on the top and bottom as separate units. This gave better results, but even then, the horizontal diameter of the cable varied from 1 in. to 2 in. more than the vertical diameter. It was felt that this variation would have no detrimental effect so the squeezing proceeded on this basis.

In the side spans the final squeezing was begun at the anchorages and progressed toward the tower. The electric pump was placed on a timber platform which rested directly on the main cables as shown in Fig. 32. The squeezers were lashed to the lower end of the platform, and were skidded up the cables with the platform by means of the hoisting engines at the anchorages.

The final squeezing began on August 19 and was completed on September 20, 1930. Concurrently with this operation, the cable bands were erected behind the squeezers, and hangers were installed for supporting the footbridges from the main cables as illustrated by Fig. 15. The storm system was also dismantled and the suspender cutting and socketing equipment set up on the top of each tower.

On completion of the squeezing operation, a survey was made to determine the actual positions of the cables in relation to the theoretical. This survey was made between the hours of midnight and sunrise to take advantage of the most favorable temperature conditions. Thermometers were placed on the cables at various points, selected to record the most probable average temperature. It must be recognized, however, that in cables of this magnitude, the actual temperature throughout the cross-section is not uniform and is, therefore, quite indeterminate. The results of the survey indicate a variation in main-span sags of 0.06 ft above, to 0.19 ft below, the theoretical. No attempt was made to check the cable sags in the side spans since, by necessity, they could not vary any appreciable amount, considering the small variation in main-span sags.

Erection of Cable Bands.—The castings of the cable bands for the main span were skidded out on the footbridges on timber sleds. The two castings forming a complete band were hoisted into position with chain blocks attached to a wood bent erected on the cables. The cable bands for the side spans were transported from the anchorages by carriages running on a pair of cables and propelled by the hoisting engines. Four castings forming two cable bands (one for each cable) were carried on each trip. They were hung from rods inserted through two of the upper bolt holes with the two halves partly surrounding the cable, but separated for clearance. After being shifted to position, the two halves were pulled together, and clamped to the cable with the permanent bolts, permitting the removal of the supporting rods.

The bolts for the bands were drawn up to 100 000-lb tension. Motors and drums, formerly used for operating the "come-alongs," furnished the

power for pulling on socket wrenches for this purpose. This was done in two stages, a preliminary tightening that drew the bolts up to the approximate tension, and a final tightening prior to the erection of the floor steel, which was checked again just before the cables were wrapped.

The tension in the bolts was checked by strain-gauges. Dynamometers were also used to measure the pull on the wrenches and were found to give very dependable results. After the final tightening, the gaps between the two halves of the cable bands were caulked with lead wool to prevent moisture from entering the cables. The caulking was omitted in the bottom of several bands at the middle of the main span to provide drainage for any water that might work into the cables.

Releasing of Footbridge Ropes.—After the hangers for supporting the footbridges and cross-bridges from the main cables had been installed, the footbridge ropes were released from the decking. It will be recalled that the footbridge sections were clamped to the under side of the ropes, which thus facilitated this operation. The release of the ropes presented no problem in the side spans, because the difference in sag between the loaded and unloaded ropes was comparatively small. In the main span, however, the difference in sag between the loaded and unloaded ropes was about 10 ft. The footbridge panels were cut loose starting at the towers and progressing toward the center of the main span. Upon reaching the quarter-points, the operation was stopped and resumed at the center of the span. When the load on the ropes had been released so that the sag had decreased an appreciable amount, the panels were kept lashed to the ropes for several points behind the point of loosening and slacked off progressively to maintain a smooth transition between the panels hung from the main cables and those still attached to the ropes.

Suspenders.—The footbridge ropes were measured for suspender lengths while hanging in free suspension, those for the side spans and the main span being connected at the tower by short pennant ropes.

A careful schedule was prepared in advance showing the length and location of the suspenders to be cut. In computing the lengths, consideration was given to the variation in stress at different points along the ropes at the time of measurement. These stresses were based on a condition of balanced horizontal components at the towers assuming the rope to hang level with the bottom of the main cables at the center of the main span, a condition which was obtained by supporting the ropes at the towers with hangers attached to rollers. The lengths were also corrected to correspond to variation in cable sags. No allowances were made for later adjustment with the use of shims.

The measurements were made at night to take advantage of more uniform temperature conditions. This also permitted suspending and reeling in the ropes during the daylight hours. Two ropes were measured each night, the ends and mid-point for each suspender being located and marked, together with its panel point number. White stripes were also painted at intervals along the rope to indicate the lay at the time of measurement. This was done to avoid changes in length resulting from possible twisting

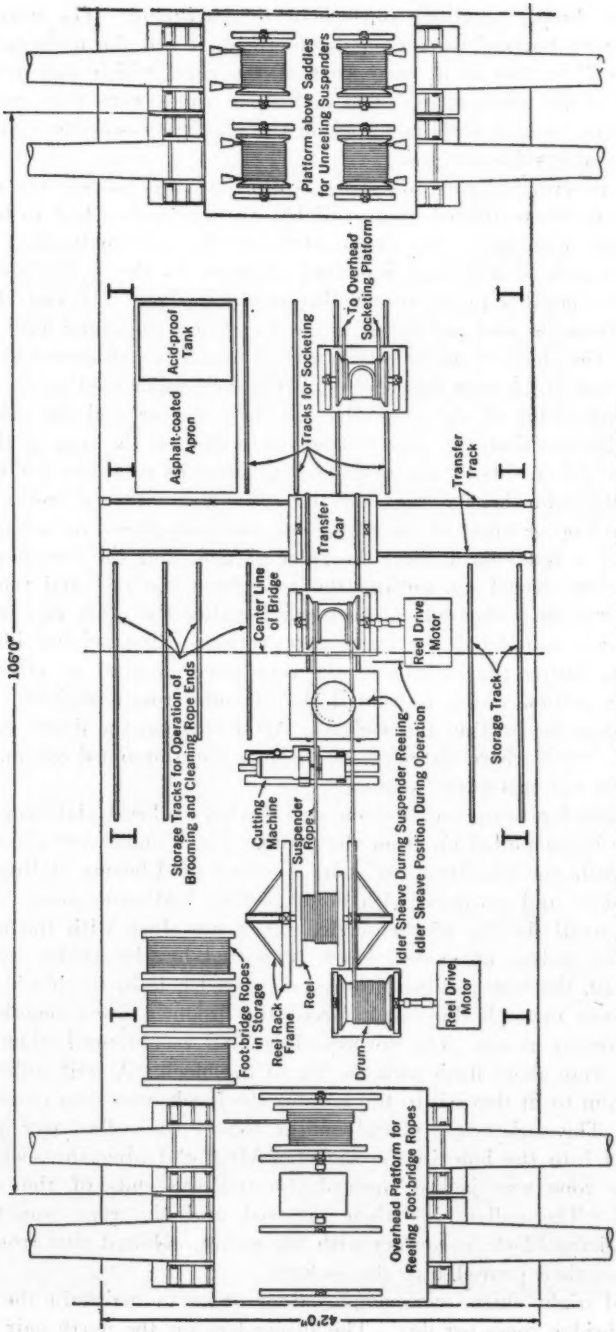

FIG. 33.—SUSPENDER CUTTING AND SOCKETING EQUIPMENT.

of the rope during handling subsequent to measuring. The next morning the ropes were lowered to rest on the footbridges and the main-span section was "burned" in two at a waste part of the rope, which was usually near the center of the main span. The main-span ropes were then reeled up at each tower on wooden reels mounted on the platforms over the cable saddles, followed by the side-span ropes.

Duplicate cutting and socketing equipment was placed on the tower tops. Due to the restricted area available, the suspenders had to be handled on reels (see Fig. 33). The one containing the half main-span length or side-span length of rope was mounted adjacent to the inside cable saddle with its axis parallel to the center line of the bridge. The end of the rope was taken from the reel and cut at the first end of a suspender with a hydraulic shear. The sheared end was then passed around a horizontal idler sheave back to a steel drum near the rope reel. The rope was reeled up on this drum until the mid-point of the suspender was half-way around the idler sheave. While in this position, two zinc buttons were cast on the rope at the correct distance on either side of the mid-point to serve as retainers for the clamp, which would hold the two parts of the suspender together under the main cable. The loop or bight of the suspender was then placed on a second drum mounted on a hand-car behind the idler sheave, and the two parts of the suspender were reeled up, pulling the rope from the reel and the adjacent drum. It was then sheared at the mark locating the other end of the suspender, which completed the cutting operation. Zinc seizing blocks with steel inserts fitting the strands of the rope were clamped on either side of the cutting points, which prevented the strands from untwisting and also served as stops for setting the sockets. After cutting, the drum was shifted to a storage track where the wires of the rope were broomed out and cleaned with acid in the customary manner.

The suspenders were socketed on an elevated enclosed platform, the ends of the rope being pulled up from the drum. These ends were clamped separately beneath the platform, to short sections of I-beams sliding between vertical guides and actuated with double-acting hydraulic-jacks. Each end was raised until the top of the seizing block was flush with the top of the guides. Two halves of a steel block supported by the guides were bolted together, with the rope passing through a tapered hole in the block. Jacking the rope down through this block forced the broomed wires together into a compact circular group. The downward jacking was stopped when the ends of the wire were about flush with the top of the block. A split collar, tapered on the bottom to fit down into the hole in the block, was then placed around the wires. The socket was placed on the top of the collar over a thin lip which fitted into the hole in the socket. After clamping the socket to the guides, the rope was jacked upward, forcing the ends of the wire into the socket. The collar was then removed and the rope was raised to bring the seizing block in contact with the socket. Melted zinc from electric furnaces was then poured into the sockets.

Day and night shifts were employed in order to maintain the schedule of two footbridge ropes per day. The suspenders for the north pair of cables

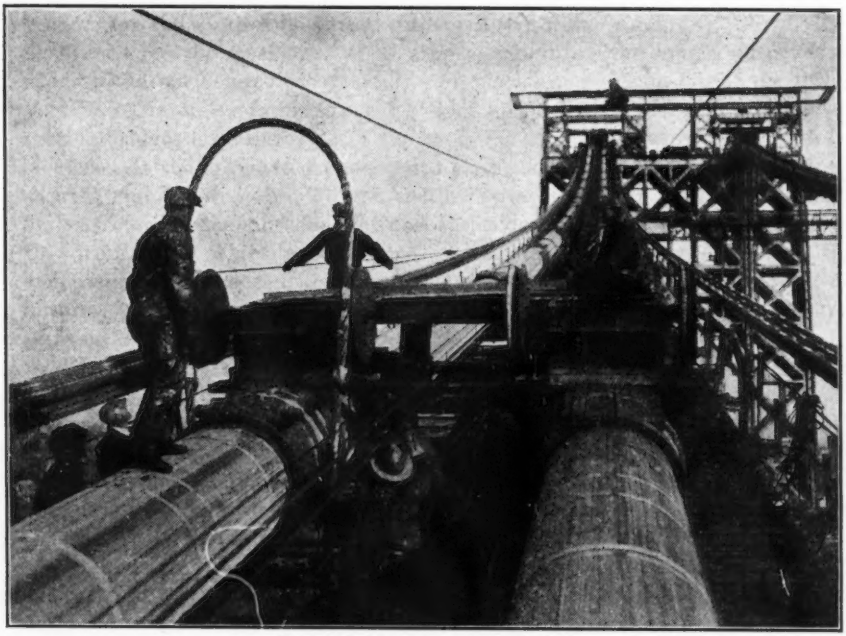

FIG. 34.—ERECTING SUSPENDERS.

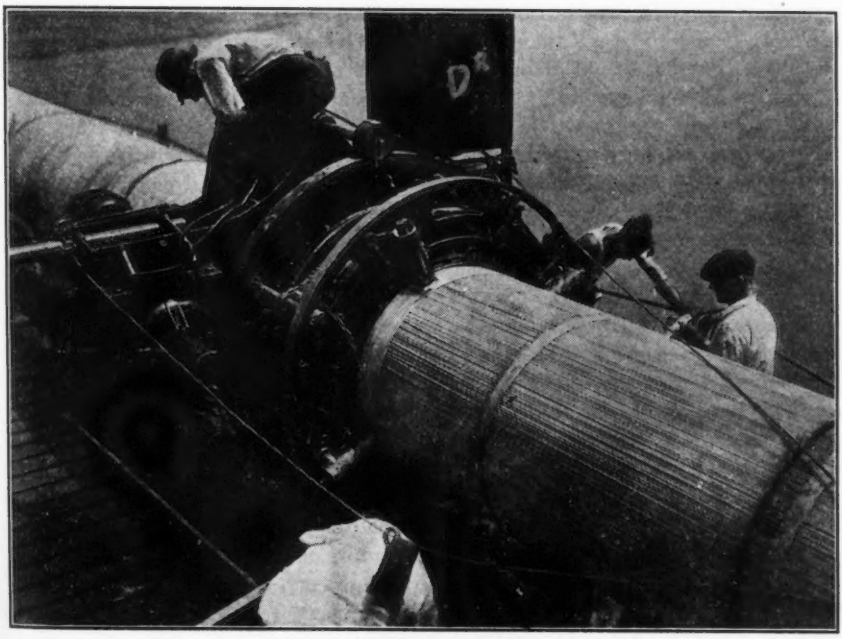

FIG. 35.—WRAPPING MACHINE.

were cut from the ropes supporting the south footbridge followed by the suspenders for the south cables from the north footbridge ropes.

Erecting the Suspenders.—The drums containing the longer suspenders, which could not be erected by hand, were placed in racks on top of the tower saddles. The ends of the suspender were allowed to unreel and slide down the footbridges on either side of the cable on which the suspender was to be erected. At the designated panel point the ends were placed over wood-lined sheaves and lowered through the footbridge as shown in Fig. 34. Manual brakes retarded the unreeling of the suspender until the bight was free of the drum, after which a wire rope leading to the hoisting engine at the anchorage, and connected to a bridle clamped to the suspender below the zinc buttons, was used to control the movement of the suspender and lower it into position.

Wrapping the Cables.—The main cables are protected from the weather by a layer of galvanized wire wrapped around the cables between bands. At the bands, the wrapping terminates in recesses machined at the ends of the castings. Water-tight connections are made by caulking with lead wool.

The wire was applied under tension in such a manner as to insure close contact between the turns. A heavy coat of red lead paste was applied just in advance of the wrapping and one coat of red lead paint and two coats of aluminum paint were applied afterward. This method produces an effective seal around the cables and has long been adopted as being the most satisfactory way to protect parallel wire cables.

In order to prevent the wrapping wire from becoming loose due to lengthening of the cables, it is desirable to postpone the wrapping until all, or at least the major part, of the dead load of the bridge is in place. For this reason the wrapping operation did not start until July 9, 1931, after the concrete roadway and sidewalk slabs were in place. It was completed on October 1, 1931.

The wire was placed by two types of machine under a tension of 400 to 500 lb as nearly as could be estimated and maintained. One applied the wrapping at the rear end as it progressed, and the other applied it at the forward end; the first, a "pusher" type, operated directly on the cable ahead of the wrapping, and the second, a "puller" type, moved along on the wrapping. Except for the direction of rotation, the two machines were essentially identical and were adopted to eliminate all hand-wrapping, the intent being to wrap approximately one-half of each panel with a "pusher" machine and then complete the panel with the "puller" machine. In actual service, however, the latter did not prove satisfactory as there was a tendency to injure the wrapping while sliding over it. Furthermore, the "puller" machine interfered with the inspection of the freshly wrapped area since it had to advance approximately 5 ft before exposing the wrapping on top of the cable; thus, if any defects existed it was necessary to unwrap for a distance of 5 ft in order to correct them. Accordingly, the "pusher" machine was used wherever possible, the remaining part in each panel being wrapped by hand.

Some difficulty was experienced, mainly in the side spans, with loose wrapping over a flat strip on top of the cable. This difficulty was overcome

by filling out the flattened area with several layers of sheet lead. These sheets were of varying width and were beveled along the edges. A strip of galvanized iron was placed on top of the lead to keep the wrapping wire from pressing into it, which would have prevented the necessary crowding action between the turns of wire.

Four machines of each type were provided, one of which is shown in Fig. 35. They were assembled on sheet-steel saddles about 5 ft long. The saddles were shaped to bear directly on the cables, a cast-steel yoke being bolted to one end. From the yoke were hung two electric motors, one on either side, with their shafts slightly above the horizontal axis of the machine. Reversible motors of 10-hp capacity, capable of driving the machine at 50 rpm, were first used, but later were replaced with 15-hp motors rotating at the same speed. An auxiliary motor operating through a "Bendix" unit and reduction gears to one of the main motors supplied a slow-speed drive for starting or wrapping the cable adjacent to the band. The motors were geared directly to a ring gear surrounding the cable at about the center of the saddle. The ring gear ran in a groove machined in a bronze bearing bolted to the saddle. The wrapping wire was carried on two reels on either side of the ring gear. These reels also ran in grooves in bronze bearings bolted to the saddle.

Band-brakes attached to the gear and rubbing on the adjacent flanges of the reels served to rotate the reels for loading with wire and also to produce the tension in the wire for wrapping. These brakes were adjustable for regulating the wire tension. They could also be released by a lever device. Supplementary brake-shoes, attached to the saddle, could be tightened to bear against the reels for unwrapping. A flyer ring, mounted in a bronze bearing at the other end of the saddle, was connected to the ring gear by two notched plates bolted to the ring and fitting over lugs cast on the side of the gear. These had to be removed each time the reels were loaded with wire. Four fingers were evenly spaced on the outside face of the flyer ring. These were made in the form of radial plungers sliding in housing bolted to the ring. The plungers were fitted with steel shoes that bore against the last few turns of wrapping wire. Lips, extending down from one edge of the shoes and just clearing the cable wire, produced a crowding action that forced each turn of wire in contact with the previous turn. Coiled springs with screw adjustment bearing against the plungers, kept the shoes in contact with the wrapping wire.

The wires were led from the reels on opposite sides of the machine passing over brass guides and through holes in the gear and flyer ring to the cable. The overturning effect on the machine due to the tension in the wires was resisted by pipe outriggers bearing on the adjacent cable. The reels, gear, and flyer ring were made in two sections, bolted together. They were separated each time the machine was shifted to a new panel.

Wrapping started at the center of the main span and at the anchorages and progressed toward the towers. No automatic mechanism was provided for advancing the machines along the cable as it was felt that the crowding action of the shoes bearing against the wrapping wire would be sufficient

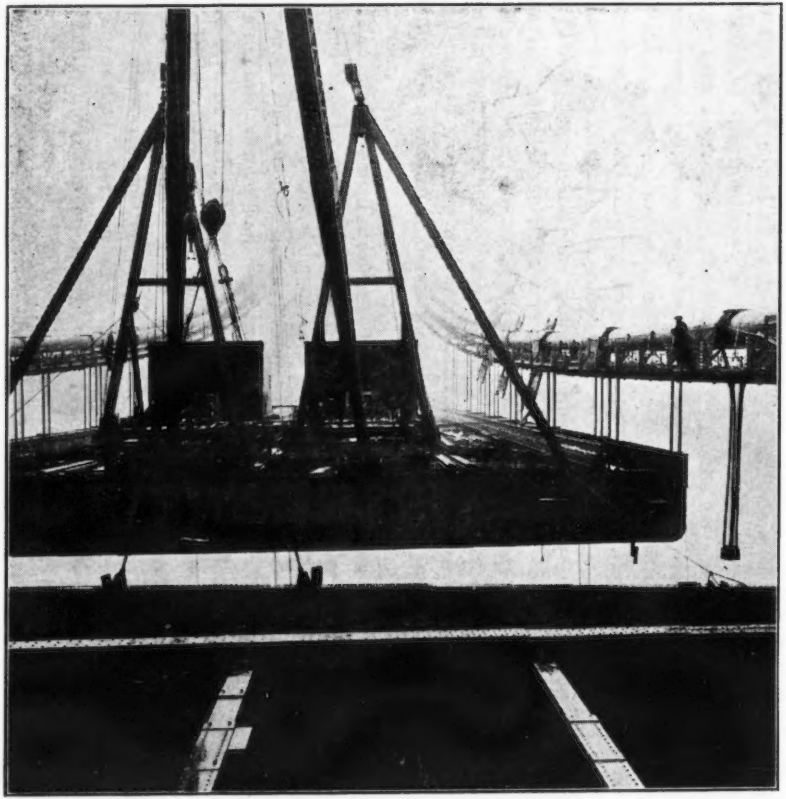

FIG. 36.—MAIN-SPAN TRAVELER, NEW JERSEY SIDE.

FIG. 37.—VIEW SHOWING FLOOR ERECTION.

to move the machine ahead. It proved necessary, however, to provide some auxiliary means to accomplish this. Experiments were made with various systems of counterweights, but the method that proved most satisfactory was simply to pull the machine ahead with a hand-winch fastened to the cable from one panel to two panels in advance of the wrapping. Manila-rope snubbing lines working against the winches were used to prevent the machines from moving in jumps.

The wrapping wire was shipped to the site on the reels used for the cable wire, which were fitted with axles and were mounted in racks on the roadway at the center of the main span and adjacent to the towers. The wire was led from the reels up to the footbridges and thence along the footbridges to the point of wrapping, those on the wrapping machines being filled from the most convenient source. The reels were rotated in the same direction while being filled as the machine rotated in placing the wire. Portable, electric, butt-welding machines were used to splice the wire.

Floor Steel Erection

General Procedure.—On September 12, 1930, while the contractor for the cables was engaged in final cable compacting operations, the steel contractor began to erect the floor steel of the towers. A Chicago boom was assembled on the lower portal of each tower, and the floor steel within the tower was erected by this means. The travelers for erecting the floor of the main span were then assembled at the towers. When the suspender ropes were in place these travelers were ready to go forward with the erection of the steelwork of the main span, proceeding from the towers toward the center of the river, hoisting the steel directly from car floats anchored in the river, and moving ahead as each panel was completed.

The side-span travelers were assembled at the towers after the main-span travelers had advanced a few panels. Because of the location of the side spans over land, the materials for these sections were hoisted from car-floats to the floor level on the main-span sides of the towers, placed on hand-cars, and pushed over the previously erected side-span steelwork to a point from which the material could be picked and placed in position by the side-span traveler.

The Main-Span Travelers.—Two **A**-frame derricks, with cable back-stays, placed side by side, made up each main-span traveler. The derricks of the New Jersey traveler (shown in Fig. 36) had been used by the contractor to erect the Cooper River Bridge, at Charleston, S. C., and those of the New York traveler had been used in erecting the Outerbridge Crossing over the Arthur Kill to Staten Island, New York, and the Mt. Hope Bridge, in Rhode Island. They were entirely different, therefore, as to detail.

The derricks in Fig. 36 were link-connected together at their sills, but were otherwise entirely independent of each other. The **A**-frames had a comparatively short spread at the sill (18 ft 5¼ in.) and, therefore, to improve stability during erection they were braced further by side guy legs of 15-in. channel sections on the north side of the north derrick and on the south side of the south derrick. The derricks were equipped with 100-ft booms.

They were mounted on trucks that traveled on rails fillet-welded to 10-in., Bethlehem girder sections laid on the transverse beams. In position for erection, the derricks were raised from the trucks, the loads being transferred to the floor-beam by means of an assembly of screw-jacks and castings under each leg. At the rear end, the derrick was lifted from its trucks by 12 by 12-in. blocking wedged between the frame and the steelwork and, at the same time, tied down by rope slings passed under the transverse **I**-beams.

Two gasoline engines (three-drum, three-speed, 150-hp), operated the New Jersey derricks. The booms were swung by block-and-fall swing lines operated from the engines, and the derricks were advanced from position to position by means of manila rope falls. The entire traveler with the engines weighed approximately 233 tons.

The traveler derricks on the New York end had a greater spread of the **A**-frame legs at the sills—about 23 ft. They were link-connected at the sills, as was the case with the New Jersey derricks. The **A**-frames were connected at the tops by two parts of 1-in. cable, and were guyed sidewise by four parts of 1-in. cable made fast to the floor-beam north and south of the traveler during hoisting operations. These guy lines replaced the struts used for the New Jersey derricks. They were mounted on trucks on rails, but were raised from the trucks by means of wedge-jacks and blocks at the sills, and blocks with wedges at the rear end of the frame before raising the loads.

Each derrick was equipped with an 85-ft boom and powered by a 4-drum, 150-hp, electric engine. These 4-drum units were those which, when coupled with units of three additional drums, had composed the seven-drum engines used in erecting the towers, as previously described. The four drums operated the main load, the auxiliary load, the boom falls, and the falls for advancing the traveler. The booms were swung by block-and-fall swing lines and a small electric motor on the traveler. The weight of the New York traveler with the engines was approximately 281 tons.

The use of electric engines for one traveler and gasoline engines for the other is explained by the fact that, although gasoline engines were preferable, only the pair for the operation of one traveler were owned by the contractor, and the purchase of additional units was not considered to be justified since the electric engines were already available at the site. The electric engines required the use of heavy conductor cables, however, which were costly and were expensive to handle in the field. The choice of the New York traveler for the use of the electric engines was dictated by the cheaper power rate on that side of the river.

Structural Steel in the Main Span.—As was the case in handling the steelwork for the towers, all the steel for the floor system was shipped from the shops to ground storage at the New Jersey terminal yards of the railroads, where the pieces were reloaded on cars in a manner that made possible their removal in the sequence required in erection. Except for the floor steel of the first panel at the New York tower, all members were hoisted from car-floats moored to barges in the river, directly below the panel being

erected. The barges were anchored in the river by lines at the four corners. These lines permitted the barges to shift a certain amount without changing the location of the anchors. When required, the anchors were changed to new positions by tugs.

The first steelwork of the suspended structure was erected in the main span at the New Jersey tower on October 28, 1930. According to the usual procedure, as followed through in erecting a typical panel of floor steel, the two booms of the traveler together raised the floor-beam, weighing approximately 62 tons. During this operation, the weight was equally distributed between the derricks by the use of a balance sheave.

While on the car-float, light scaffolds were attached near the ends of the floor-beam to give the bridgemen positions from which to work while entering the socketed ends of the suspender ropes in the seat connections provided on the floor-beam. Workmen did not ride the floor-beam during the hoisting operation, but walked out to it on 12 by 12-in. timber struts, used to space the floor-beam from the panel previously erected.

The fastening of the eight suspender-rope sockets to each end of the floor-beam was the most tedious part of the operation. Reference has been made to the lines painted on the suspender ropes during the process of measuring, the object being to permit attachment of the rope without excessive twisting or untwisting. For the longer ropes, a single turn variation from the condition under which the rope was measured may so alter the lay as to change the length by as much as $\frac{1}{2}$ in. It was highly desirable, of course, to maintain the measured length of suspender. It was intended by observation of the painted line to make sure that no twists occurred. In actual practice it was found to be exceedingly difficult to determine whether or not the long suspenders were entirely free from twists. The line could be followed for a considerable distance, but under certain conditions of light, especially in hazy weather, it was almost impossible to follow the painted line to the top of the suspender.

After the floor-beam had been set in place, each of the two derricks proceeded to operate independently in filling in the remainder of the steel required in the panel. The first members were the wind chords, which were entered in the floor-beams from below. These were followed by the stringers, bracing, and transverse beams. The bulb-beams were hoisted to the deck, but were not placed in position by the travelers. Otherwise, the entire floor steel was erected complete in one pass. Because of the relatively great mass of the cables, such a procedure was possible without producing objectionable distortions in spite of the great weight of the travelers. Saddle motions, cable bending, saddle friction, operating grades for travelers, and general as well as local floor distortions for this condition were carefully calculated in advance to be within safe and practicable limits.

The floor steel followed a curve that was concave upward until erection progressed to approximately Panel Point 38. (See Figs. 37 and 38.) At this point, the curve became flat and thereafter it was concave downward. The curvatures were much less pronounced than in other relatively lighter structures. The deformations of the main floor during the various stages of

construction are shown on Fig. 38, for a dead floor load of 8 000 lb per ft of bridge. For purposes of comparison all data were corrected to a normal temperature of + 50° F. In a few cases the corresponding theoretical profiles are plotted.

However, some difficulties were experienced because of the distortion, principally with the wind chords and the diagonal wind-bracing. The ends of the chord members are milled at right angles although the final curvature of the chords is concave downward. The secondary stresses induced by this curvature were calculated as being too small to warrant milling at the slight angle required.

As erected, the abutting chord members were in contact at the top and open at the bottom of the joint. It was impossible, therefore, to fit up the splices completely, immediately after erection; and yet, because of the action of the wind on the erected part of the floor system, it was essential that a certain minimum number of bolts be placed in order to carry the stresses safely. The desired results were obtained by fitting up the splices in the cover-plates, the remaining splice-plates being left loose. As the erection progressed, a careful check was made from time to time to determine the profile, and whenever the chords over two or three panels tended to approach a straight line, the remainder of the splice at these points was fitted up. No drifting was permitted when it would be injurious to the holes. Special drift-pins of exact diameter were driven by air hammer.

The floor-beams are framed to hang vertically in the completed bridge, rather than normal to the roadway. As erected, the chord members sloped upward away from the traveler, causing a considerable angle between chord and floor-beam connections. Therefore, when first entered from below, the chord was held up to the forward floor-beam by bolts through the cover-plate. The tilting of the floor-beam necessary to complete the connection occurred when the stringers were erected. These stringers were bolted only through the holes in the seat connections at both ends as first erected. The eccentric load from these stringers on the forward floor-beam also tipped it sufficiently to permit "making" the top holes of the stringer connections. Theoretically, the fact that the chords hinged at the rear splice-plates, 8 ft off the panel point, would cause the chords and stringers to come into the floor-beam at slightly different angles. However, the difference was not noticeable in actual erection.

Some difficulty was experienced in erecting the diagonal members of the wind-truss system. These members lie below the plane of the stringers. Accordingly, during the time that the profile of the floor followed a curve concave upward, the actual distance between the points of attachment of the ends exceeded the final amount. At times, it appeared to be almost impossible to make the end connections of both the diagonals in a panel. However, by the use of small bolts the connections were made, although the final fitting was of necessity deferred until a more favorable profile was obtained.

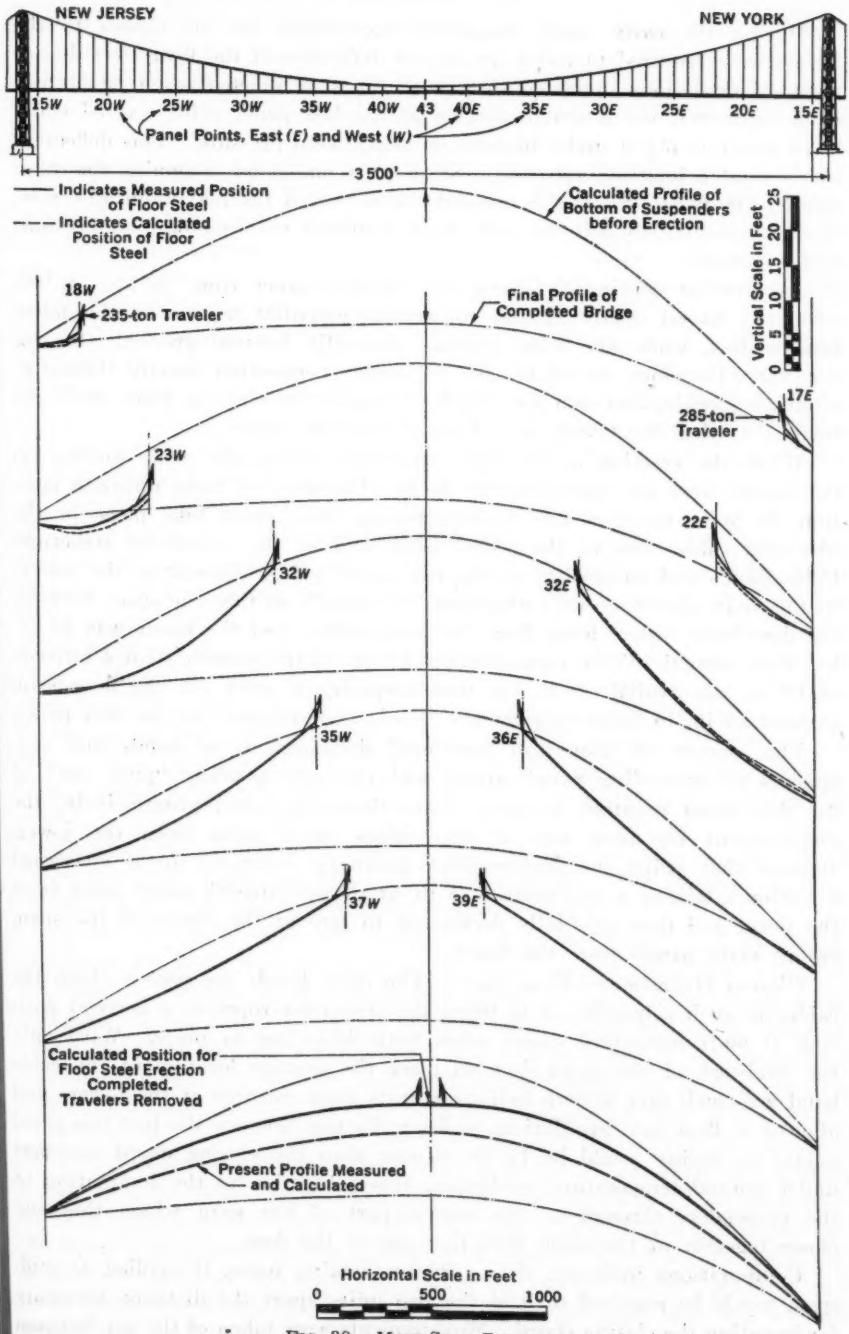

FIG. 38.—MAIN-SPAN PROFILES.

During the early stages temporary connections of the chords to the towers were required to resist the lateral deflection of the floor steel due to wind. Calculations indicated that, with the steel erected seven panels out from the tower, the lateral deflection at the last panel point erected could be as much as 13½ ft under moderately heavy wind pressure. This deflection would cause a longitudinal motion of 45 in. in one chord, assuming the other chord to be fixed. Since this amount would exceed the maximum allowable, it was necessary to bolt the end chord members to temporary connections at the towers.

As erection continued beyond the seventh panel from the tower, the computed lateral deflection and consequent potential motion at the tower became less, while the wind stresses naturally became greater. It was necessary, therefore, to release the temporary connection shortly thereafter although blocking between the chord and tower was left in place until the traveler reached the twenty-second panel from the tower.

When the erection of the floor steel was begun, the cable saddles on the towers were set approximately 23 in. shoreward of their ultimate position, in order to allow for the lengthening that would take place in the side-span cables due to the added dead load of the completed structure. Cable bands and suspenders at the first panel point adjacent to the towers in the main span were thus displaced horizontally so that the space between the floor-beam—when hung from the suspenders—and the tower was 23 in. less than normal. With normal shims of 6 in. at the bumper strut a latitude of 10 in. was available. It was thus necessary to push out the floor-beam at least 13 in. in order to erect the chords and stringers in the first panel.

The amount of shoreward horizontal displacement of bands and suspenders at succeeding panel points, and the corresponding "push out" of the floor-beam required to make connections, decreased progressively, the displacement becoming zero at the eighth panel point from the tower. Beyond that point the displacement gradually increased in a riverward direction attaining a maximum of 8 in. at the seventeenth panel point from the tower and then gradually decreasing to zero at the center of the span, twenty-eight panels from the tower.

Closing Operations—Main Span.—The cable bands are spaced along the cables in such a manner as to bring the suspender ropes to a vertical position at 60-ft horizontal spaces when both decks are in place. With only the steelwork of the upper deck in place the average location of the cable band was such that if both half-span units were cut free at the towers and allowed to float into equilibrium position the gap between the last two panel points at closing would be 1¼ in. shorter than the closing chord members under normal temperature conditions, this being due to the inclination of the suspenders throughout the central part of the span where they are closer together at the cable than they are at the floor.

Computations indicated that a 26-ton jacking force, if applied at midspan, would be required to hold the two units apart the distance necessary for inserting the closing chord. Measurements were taken of the gap between

milled faces of chord splices when steel had been erected to within three panels of closing. The New Jersey half-span unit of floor steel was then pulled toward the tower by means of turnbuckles, an amount necessary to cause an opening sufficient to allow for the length of steel to be inserted, plus the shortening of the gap due to the deflection of the two half-span units while adding the last three panels, plus an allowance for a possible change due to temperature rise subsequent to the measurement, plus a nominal $\frac{1}{2}$-in. allowance for insertion of the closing chord.

The shortening of the gap due to the profile changes subsequent to the measurement was computed to be 1 in.; and the effect of temperature would close the gap $2\frac{1}{2}$ in. for each rise of 10° in temperature.

As it developed the temperature rise was slightly more than anticipated although not enough to cause any difficulty in closure. The closing chord members were set in place with $\frac{1}{8}$-in. clearance. The last panel was erected at the center of the main span on December 29, 1930, 62 days after the erection of the first suspended steelwork (October 28, 1930). Fifty-eight panels of steelwork were erected by the main-span travelers in 40 working days, an average only slightly less than $1\frac{1}{2}$ panels per day.

The Side-Span Travelers.—The travelers for the erection of the side spans were alike. Each consisted of one of the stiff-leg derricks used on the tower traveler. Because of the comparative shortness of the cables in the side span little change in cable deflection was to be considered in erecting the steel, and for the same reason, progress in erection could be allowed to lag far behind that of the main span. Of course, the reverse would not be true; side-span steel erection could not precede the main-span steel erection. The single-boom stiff-leg derrick, while slower, was, therefore, entirely satisfactory; in fact, it was the most satisfactory type of equipment for swinging back to handle members trucked to it over the floor steel for erection in the following panel.

The derricks had 85-ft booms and were designed to have a capacity of 84 tons at 60-ft radius and 65 tons at 80-ft radius. The heaviest piece to be handled was the 62-ton floor-beam at a 60-ft. radius.

It will be recalled that the engines for the operation of the tower erection travelers were near the base of the tower on the shore side. One of the three-drum units (an electric engine of 150-hp capacity) left in its original position, was used to operate the side-span traveler. The "cat-heads," which had been used in hoisting the tower erection traveler were located at convenient points to carry the leads from the derricks to the engine. Special sheaves were also provided to support the leads above the floor steel.

Structural Steel in the Side Spans.—The steel for delivery to the New Jersey side span was brought from ground storage in car-floats which were moored on the river side of the tower foundation piers. The remaining stiff-leg derrick of the tower traveler was set up within the portal and on the south side of the roadway at the river face of the tower, one leg being replaced by a short connection to the tower steel. In this position, the boom reached over the side of the floor steel on the main span and hoisted members to the deck level. There was just sufficient space to manipulate the

main floor-beams between the face of the tower and the first group of suspender rope. The members were placed on trucks, operating on the bulb-beams as rails, near the north side of the roadway, and were then carried to a position in the last panel erected, within reach of the traveler boom. (See Figs. 39 and 40.)

The arrangement for hoisting the steel to the roadway level on the New York end was somewhat different. It will be recalled that the New York tower foundations are back a short distance from the water's edge, necessitating a temporary mooring wharf for the car-floats. Accordingly, the stiff-leg derrick for hoisting side-span materials was skidded to position in the second panel of the main span, from which position it could reach over the side of the floor steel and hoist the steel members to the floor level for trucking to the traveler.

The floor steel was erected in the same order as was followed on the main span, although somewhat different conditions were to be met. The side-span travelers practically always worked on a down-grade ahead. As in the main span, the floor-beams were framed to hang vertically in the completed bridge and, therefore, required tipping forward to permit matching holes in the connections between the chords and the floor-beams.

At the beginning of erection the cable bands and suspenders at the first panel adjacent to the tower in the side span were approximately 23 in. shoreward of their normal position because of the setback of the tower saddles as previously explained. The displacement of cable bands and suspenders shoreward was, of course, progressively less at each panel point toward the anchorage until the last was only 3 in. shoreward horizontally. All these conditions were anticipated and met by pulling the floor-beams back toward the traveler to make the chord connections, and it was necessary to lash the chords to the tower to avoid pulling the first panel stringers off their seats on the towers.

As in the main span, temporary connection with blocking was used during the erection of the first few panels of floor steel to resist the longitudinal motion due to lateral deflection of the floor steel under wind pressure.

Erection operations in the side spans were slower than those of the main span, two or three days being required for each panel. The New Jersey side was completed on January 8, 1931, and the New York side on January 21, 1931.

Tower Deflections, and Positions of the Cable Saddles.—Variations in position of the tops of the towers for various conditions of loading are shown in Table 2. Typical average conditions, which include the maximum range during cable spinning and erection of the floor steel are shown. For simplicity, the motions are given for the New Jersey tower only. Except for slight discrepancies that are unexplainable, these motions agree reasonably well with values calculated in advance.

The effect of the footbridge ropes as erected, but not adjusted, was to cause a slight riverward deflection. This was caused by the fact that tensions in side-span and main-span ropes were approximately equal at the time, resulting in an unbalanced riverward H-component. Considerable

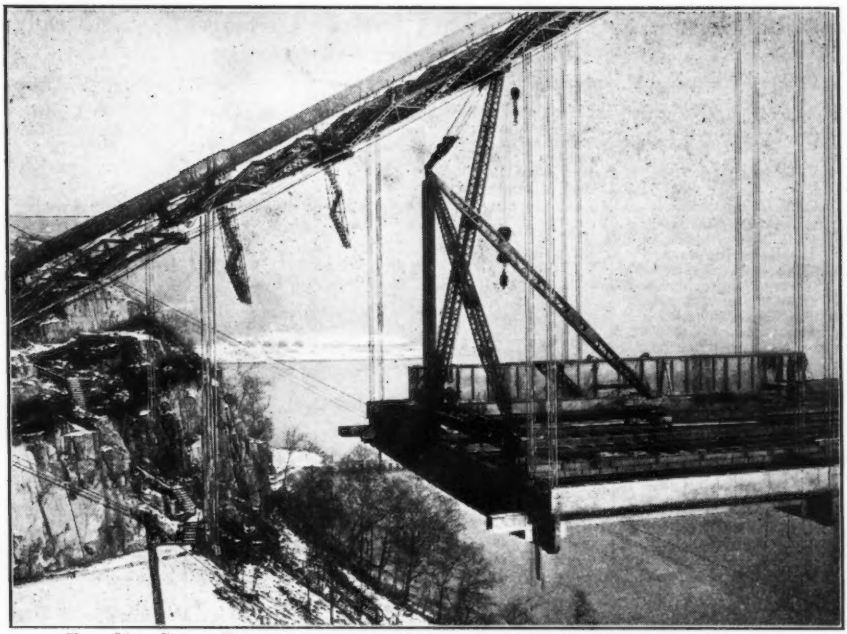

FIG. 39.—STEEL ERECTION IN SIDE SPANS; FLOOR-BEAM READY FOR PLACING.

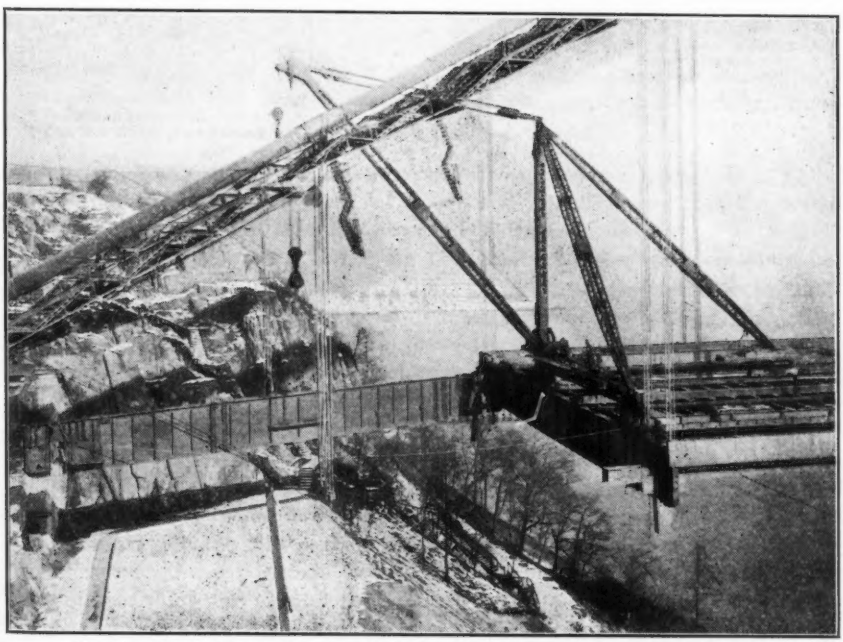

FIG. 40.—STEEL ERECTION IN SIDE SPANS; FLOOR-BEAM BEING SET IN PLACE.

TABLE 2 — DEFLECTIONS OF NEW JERSEY TOWER DURING CABLE SPINNING AND ERECTION OF FLOOR STEEL

Date	Temperature, in degrees Fahrenheit (Normal, 50° F.)	Average deflection from vertical toward the river, in feet	Average position of cable saddles relative to the top of the tower (toward the shore), in feet	Remarks
July 1, 1929.....	70	0.00	1.89	Normal unloaded tower.
August 8, 1929.....	84	0.03	1.89	Footbridge ropes erected, but not adjusted.
September 11, 1929..	70	0.16*	1.89	Footbridge ropes adjusted.
October 22, 1929....	70	0.47	1.89	Footbridges completed.
November 27, 1929..	38	0.56	1.89	†First set of strands in permanent position for Cable *C* and spinning position for other cables.
December 12, 1929..	26	0.51	2.00	First set of strands in Cables *A*, *B*, and *C* adjusted; partly adjusted in Cable *D*.
September 30, 1930..	60	0.32	2.00	Cables completed and footbridge ropes released.
November 13, 1930.	60	0.39	2.00	Four panels of main-span steel erected adjacent to each tower. No side-span steel erected.
November 18, 1930..	54	0.10	1.65	Five panels of main-span steel erected adjacent to each tower.
December 11, 1930..	42	0.25	1.65	Eighteen panels adjacent to New Jersey tower and sixteen panels adjacent to New York tower in main span; three panels in New Jersey side span.
December 11, 1930..	42	0.06	1.40	Eighteen panels adjacent to New Jersey tower and sixteen panels adjacent to New York tower in main span; three panels in New Jersey side span.
May 13, 1931.....	50	0.36	1.40	Floor steel completed.

* Deflection toward the shore.
† The four cables were designated *A*, *B*, *C*, and *D*, from south to north.

tightening was required for adjustment in the side-span ropes, which pulled the top of the tower shoreward. The original intention was that the towers would be vertical for this condition at normal temperature. It will be observed that the greatest deflection occurred with the first few completed strands in their spinning positions, the tower being gradually drawn more nearly vertical by the successive adjustment of completed strands. Furthermore, the tendency of the cables to cause excessive riverward deflections during the erection of the floor steel was offset by adjustment in the position of the cable saddles. The changes in position were made by jacking, the position being maintained by temporary blocks inserted between the river side of the saddles and the steelwork of the tower.

When the structure had been completed to its present stage, the saddles were jacked to a position $7\frac{1}{8}$ in. shoreward of the center line of the top of the tower for the New Jersey tower and $5\frac{13}{16}$ in. shoreward for the New York tower, these positions being such that the towers would assume vertical positions for normal temperature under the dead load of the completed upper deck. The saddles are maintained in these positions by permanent steel blocking.

Construction Schedule

A definite construction program, with dates to be met, was established in the contracts. Certain modifications were made in the original schedule at the time the cable contractor changed from his original plan of successive erection to that of simultaneous erection.

According to the schedule as revised, the erection of the steelwork of the two towers was to be completed by June 1, 1929, to such an extent as to permit the erection of grillages, saddles, and cable falsework, and all work on the towers was to be completed by September 1, 1929. Cable construction, except wrapping, was to be completed by November 1, 1930, in order that erection of the floor system might begin on that date, and the floor system was to be completed by May 1, 1931, to such an extent as to permit a start to be made on the construction of the concrete deck. All work on the steel floor system was to be completed by July 31, 1931.

The dates of the construction program are cited in order to indicate how closely the advance program was adhered to on work of such magnitude. Assembly of erection equipment was begun on the foundation of the New Jersey tower on May 1, 1928, and on the New York foundation, about July 15, 1928. The towers were completed, ready for cable operations, during June, 1929. At that time the contractor for the cable construction had made a beginning on the installation of his equipment. Grillages under the saddles on top of the towers were erected on the towers early in July, and the erection of the spinning equipment, including the steel superstructure on the towers, and the footbridges, proceeded during the months of July, August, and September. Cable spinning was begun October 21, 1929, and was completed August 8, 1930, after which the suspender ropes were placed in time to permit the erection of the first suspended steelwork on October 28, 1930. Nine weeks later, December 29, 1930, the last panel of main-span steelwork was in place. The side-span floor steel erection, which proceeded more slowly, was completed on January 21, 1931.

Contractors and Personnel

The towers and floor steel were supplied and erected by the McClintic-Marshall Company at a cost of $10 823 000. For the contractor, David S. Gendell, Jr., M. Am. Soc. C. E., Manager of Erection, was in charge, assisted by Mr. A. S. Halteman, as Resident Engineer. Mr. H. G. Reynolds was Foreman for the work on the New York side and Mr. W. B. Fortune on the New Jersey side.

The cables, suspenders, and anchorage steelwork were fabricated and erected by the John A. Roebling's Sons Company at a cost of $12 193 000. Mr. C. C. Sunderland was Chief Engineer for the Contractor, and Mr. C. M. Jones, Assistant Chief Engineer. W. E. Joyce, Assoc. M. Am. Soc. C. E., served as Resident Engineer and Mr. W. R. Coppage as Superintendent.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

REPORTS

METEOROLOGICAL DATA

PROGRESS REPORT OF SPECIAL COMMITTEE[1]

To the Board of Direction,
American Society of Civil Engineers,
New York City, N. Y.

Gentlemen.—Your Committee appointed in April, 1931, to "give thought as to how the United States Weather Bureau could be made of greater service to engineers" submits the following report.

A brief statement as to the background for this study is first given, including the Committee's recommendations for improving the scope and quality of the technical material emanating from the Bureau. Then follows the report proper, giving under appropriate section headings a résumé of the history and work of the Committee, with a more extended discussion of the factual findings on which its recommendations are based.

Summary, Including Recommendations

The Committee sent out a questionnaire to 204 members of the Society and others who were interested in the subject, and received 141 replies. It also made a number of investigations itself. From the replies to the questionnaire and the studies carried on by the Committee, the following conclusions are drawn:

1.—Meteorological data, as collected and published by the U. S. Weather Bureau, are essential in the economical planning, construction, and operation of many types of public and private enterprises and improvements, in which vast sums of money have been, and are, being invested, and such data are widely used by engineers engaged in this work.

2.—A widespread feeling of dissatisfaction exists in the Engineering Profession in regard to:

(*a*) The location and distribution of stations.
(*b*) The quality of records.
(*c*) The manner in which the data are published.

Much of this dissatisfaction and the resulting criticism could be eliminated and better service rendered by the Bureau at little or no increase in cost or appropriations.

3.—Research in meteorology by the Bureau, particularly along the lines which are used by the Engineering Profession, has not kept pace with the

Note.—Discussion on this report will be closed in **April, 1933**, *Proceedings*.
[1] To be presented at the Annual Meeting, January 18, 1933.

growing importance or utilization of this type of data, nor with research in other lines of science, either pure or applied.

4.—The historical background of the Bureau and its present practices indicate that the activities of the Bureau in the collection and dissemination of this type of data have:

(*a*) Developed into more or less routine or mechanical procedure.

(*b*) Been without consideration of specific needs or new conditions as they arise.

(*c*) Been largely for the purpose of short-time forecasting.

5.—Too much centralization of authority, responsibility, and initiative in the Office of the Chief of the Weather Bureau exists at present to allow the development of proper initiative, morale, and activities on the part of the personnel of the Bureau.

6.—When compared with other governmental organizations of similar character, the personnel structure of the Bureau exhibits:

(*a*) A decided lack of educational background among its employees.

(*b*) An excessive proportion of employees in the lower grades, with a resulting low schedule of compensation.

7.—Appropriations for the support of the general activities of the Bureau (not including aerological work) have maintained a reasonable scale of increase in the past.

8.—Salary ranges for various Civil Service classifications in the Bureau appear to be quite fair, when compared with other public technical and scientific organizations, but the under-classification of employees in such grades is apparent, and does not develop an organization which can attract to it, and maintain within it, the type of men which its importance justifies; nor does such under-classification develop a spirit or morale within the employee ranks, which should exist in an organization of this type.

9.— The distribution of stations of the Bureau, both geographically or in respect to altitude, is not such as to give a proper presentation of climatological characteristics of the entire country, there being an excessive concentration of stations in some localities and a like scarcity in others.

10.—A lack of inspection of location and exposure of recording instruments at co-operative stations has resulted in the collection and publication of records for many localities that are either erroneous in themselves, or not indicative of true climatic characteristics of the locality which they represent.

11.—The practice, particularly at regular stations, of placing recording instruments for precipitation, wind, and temperature on roofs of buildings, gives records and results which are not indicative of actual climatic conditions at the station.

12.—The practice of changing, from time to time, the elevation above ground and the location of recording instruments, makes continuous records of stations where such changes have been made, unreliable as indices of climatic phenomena and their variation. When such records are published with no informational backgrounds as to such changes, they are actually misleading.

The Committee is still giving thought to the many excellent suggestions made in the replies to the questionnaire. In order to develop greater service on the part of the Weather Bureau to the Engineering Profession at the present time, the following recommendations are made:

1.—That a co-operative program be developed with other Federal departments, and with State, county, and city agencies, railroad companies, and other public utilities, which program will allow an extension of stations into areas not now served, an increase in the present number of stations where needed, and other necessary activities, at nominal cost to the Bureau.

2.—That a publication be issued in bulletin form giving information pertaining to exposure, changes in elevation and location of recording instruments, and relative reliability of records of precipitation, wind, and temperature for all stations for which records are now, or have been heretofore, published.

3.—That co-operative stations be inspected, whereby improper exposure of instruments may be corrected and incorrect location of stations may be remedied, so that observations at such stations will be indicative of climatic phenomena in the locality.

4(*a*).—That when the elevation or location of a station is changed in the future, simultaneous observations be taken at the original and subsequent stations for a sufficient time to allow the determination of a relationship between the results at the two stations, such relationship to be published in all succeeding summaries.

4(*b*).—That stations be located at ground level wherever practical, and where not practical, that parallel observations be carried on at a near-by ground-level location.

5.—That a program of research be initiated by the Bureau which will allow the development of a correcting factor or relationship that can be applied to records obtained from instruments exposed above ground level, bearing in mind that while such a factor or relationship will not be of an exact nature, it nevertheless will be exceedingly valuable in the utilization and analyses of records obtained at elevated stations.

6.—That a program of co-operative research with the many scientific institutions throughout the country be initiated.

7.—That the Bureau be re-organized so as to allow for a grouping of the present sixteen administrative branches into from three to five divisions, and also to allow more authority, responsibility, and initiative to be exercised by local officials.

8.—That a policy of promotion be adopted within the Bureau which, in the near future, will:

(*a*) Bring into the intermediate grades of the Bureau a larger proportion of the personnel than now exists, these grades to be filled only by men with proper background and aptitude for work in pure and applied science; and if a sufficient number of men do not exist in the lower grades of the Bureau, such positions to be filled by men outside the present service.

(*b*) Bring into the lower grades of the Bureau only men who have education and training of a character necessary to develop the Bureau into the proper type of scientific organization.

9.—That upon the retirement of the present Chief of the Weather Bureau and the appointment of a successor, the President be urged, in the selection of such successor, to give the most favorable consideration to the man who measures up to the following qualifications:

"First, the Chief of the Weather Bureau should be a man with experience in administration of organizations of magnitude.

"Second, a broad fundamental scientific training should be a requisite, and the experience of the Chief should have been along the lines of either pure or applied science.

"Third, specialized training or experience in the field of meteorology is desirable, but is not so essential as to be the determining factor.

"Fourth, the Chief should combine the rare qualities of mature judgment and progressiveness with an appreciation of the potential opportunities of service by the Weather Bureau to agriculture, commerce, industry, and engineering.

"Lastly, the Chief must be a courageous and, at the same time, diplomatic leader, who will release latent abilities now bound by archaic tradition."

The Committee feels that its activities should be continued and offers the following suggestions as to a future program:

1.—A further study of the many excellent suggestions developed by means of the questionnaire.

2.—The development, under the sponsorship and general guidance of the Committee, of a program of research work in engineering and scientific schools.[1a]

I.—Historical

In order to obtain a proper perspective of the work of the United States Weather Bureau, the following brief historical sketch on the development of weather observations and of the Bureau is presented.

Records of meteorological observations made by the Ancient Greeks, Romans, and Hindus prior to the Christian Era still exist.[2] Aristotle's treatise on winds written during the Fourth Century, B. C., formed the basis of every standard textbook on meteorology and climatology which was published until the commencement of the Sixteenth Century.

The first organized attempt to systematize and correlate weather observations[3] was made in 1654, when Ferdinand II, Grand Duke of Tuscany, secured the co-operation of several observers in Italy and adjacent countries and distributed meteorological instruments and forms. Records were kept

[1a] *Civil Engineering*, September, 1932, p. 595.
[2] "Our Weather," by Charles Fitzhugh Talman, N. Y., Reynolds Pub. Co., p. 67.
[3] *Loc. cit.*, pp. 213, 215.

by this group until 1667, although few have been preserved. The next organized step in obtaining systematic records was started by the Meteorological Society of the Palatinate, founded in Mannheim, Germany, in 1780. Thirty-seven stations were established throughout Europe and two in the United States, and daily observations upon meteorological phenomena were made. The French Revolution in 1792 disrupted this effort. In 1820, H. W. Brandis, a German physicist, initiated the first attempt at compiling synchronous records of daily weather phenomena, observations made by the Palatinate Society for the year 1783 being compiled.

What is believed to be the first regular weather record maintained in the United States[4] was that kept by the Rev. John Campanius at Swedes Fort, near Wilmington, Del., during 1644-45. Meteorological records near Boston, Mass., during 1729-30 were kept by the Hon. Paul Dudley, Chief Justice of Massachusetts. Thermometer records at Charleston, S. C., were commenced by Dr. John Lining in 1730, and a more extensive series of observations were maintained by him from 1738 to 1750. Records at Cambridge, Mass., were maintained by Professor John Winthrop, of Harvard, from 1742 to 1748. Thomas Jefferson and James Madison maintained a series of contemporaneous records at Monticello and Williamsburg, Va., respectively, during the period, 1772 to 1777.

Josiah Meigs, Commissioner of the United States General Land Office,[5] established tri-daily observations at all Land Offices in 1817, this being the first attempt on the part of the Federal Government to make systematic observations on climatological phenomena. In the following year, the Surgeon-General of the Army issued an order requiring all surgeons in Army hospitals to keep observations on the weather. In 1825, the State of New York, and, in 1837, the State of Pennsylvania, initiated State-wide collection of weather data. In 1841, the United States Patent Office commenced the collection of such data upon a broader scope, and, in 1847, the Smithsonian Institution inaugurated a system of organized observations. The first published weather forecast based upon simultaneous telegraphic observations was inaugurated in 1849 by Professor Henry, of the Smithsonian Institution, and these forecasts were continued until 1861.

The plans for the present Weather Bureau were originated in part by Professor Cleveland Abbé, Director of the Mitchell Astronomical Observatory, at Cincinnati, Ohio, during the period, 1868-70. In 1869 the suggestion was made to the Secretary of War by Col. H. A. Myer, then head of the United States Signal Service, that a scheme of weather reports and storm signals be instituted. About that time Professor I. A. Lapham, a scientist of Milwaukee, Wis., and one of Profesor Abbé's co-operating observers, addressed a petition to Congress asking for the establishment of a National Weather Service. Representative H. E. Paine, of Wisconsin, introduced a bill providing for such service as a part of the Military Signal Service,

[4] "The Climates of the United States," by Robert DeCourcey Ward, Bost., Ginn & Co., 1925, p. 3.

[5] "The Weather Bureau," Service Monographs of the U. S. Government, No. 9, Inst. of Governmental Research, N. Y., D. Appleton & Co., 1922, p. 3.

which bill was approved February 9, 1870. (16 Stat. L., 369.) The bill provides:

"For the taking of meteorological observations at military stations in the interior of the continent and at other points in states and territories of the United States, and for giving notice on the northern lakes and at the sea coast, by magnetic, telegraph and marine signals, of the approach and force of storms."

This wording would indicate that the original purpose in the establishment of the Weather Bureau was that of forecasting weather conditions for lake and ocean commerce.

While the Signal Service carried on this work, general administration and observations were placed in the hands of officers and enlisted men of the Service, but the higher technical work was directed and performed, in a large measure, by civilians. Soon after the creation of the Weather Service within the Signal Service, its benefits became widely appreciated by commercial and agricultural interests in interior districts, and, in 1890, Congress passed an Act (26 Stat. L., 653) transferring the Weather Service of the Signal Corps to the Department of Agriculture, effective July 1, 1891.

The observation stations of the Signal Service were located throughout the United States, in principal cities, at important lake and seaports, and at military posts, such locations being selected chiefly on account of their general advantage for forecasts and storm-warning purposes rather than for climatological reasons.[6] Observations were made of actual maximum and minimum temperatures, air humidity, barometric pressure, rainfall, direction and velocity of wind, kinds of clouds and direction of their movement, and the state of the weather.

The first observations made by the Signal Service commenced November 1, 1870. Until the end of 1884 two sets were taken, one for telegraphic purposes, consisting of simultaneous observations at all stations, and the other consisting of observations for general climatological purposes. After 1884, special observations for climatological purposes were discontinued because of the general introduction of self-registering instruments for recording climatic and other scientific data. Until 1881, official weather forecasts were made only at the Central Office in Washington. By 1890, officers were assigned to St. Paul, Minn., and San Francisco, Calif., to make forecasts for their respective vicinities, and, at present, there are five principal forecast districts, while nearly every local official is authorized to make local forecasts.

The vessel weather service of the Northeast Pacific Ocean[7] was inaugurated in 1921, and has been developed to a point where an average of fifty reports are received daily from ships off the Pacific Coast of North America. This has resulted in marked improvement in forecasts for Pacific Coast and Mountain areas.

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[6] "The Weather Bureau." Service Monographs of the U. S. Government, No. 9, Inst. of Governmental Research, N. Y., D. Appleton & Co., 1922, p. 6.

[7] *The Monthly Weather Review*, August, 1929, p. 334.

In 1881, a system of weather observations and reports throughout the cotton belt was initiated, as were special warnings for the benefit of the sugar interests. In 1888, the cold wave flag and many other signal devices were introduced, and, in 1902, the daily forecast of weather observations to vessels at sea was inaugurated.

From time to time aerological observations by means of balloons and kites were carried on by the Bureau, and special meteorological studies in allied fields have been undertaken in a scattered way. The official publication of the Weather Bureau, *The Monthly Weather Review,* was commenced on January 1, 1873, and has continued until the present time.

A more detailed description of the present activities and organization of the Bureau is given under Section V.

II.—Causes of Formation of the Committee

As the field of engineering advances, more and more data, both as to quantity and kind, relating to natural phenomena become necessary for the economical design, construction, and operation of engineering enterprises. Greater refinements arise from new theories and practices developed, lower factors of safety are used, which, in turn, call for greater accuracy in making engineering estimates and studies. In the field of civil engineering during the past few decades an increasing use has been made of meteorological data of various types, and now such data are essential for the planning, construction, and operation of many types of private and public enterprises and improvements. The data mainly used in these are of four classes:[8] Precipitation; temperatures; wind movements; and evaporation.

Precipitation.—Records of the amount and variation of precipitation over periods of months and years become necessary in the making of water supply estimates for new hydraulic projects and for the enlargement of existing ones. Stream-flow records upon which water supplies are based are, in most cases, of insufficient length to determine the yield of watersheds over succeeding wet and dry cycles, and reliance must be had on precipitation records, which are usually of longer duration, for the extension of existing stream-flow records. Studies of ground-water supplies require a knowledge of precipitation occurring in each individual storm for the determination of soil moisture and evaporation. Data pertaining to the depth and water content of snow are becoming exceedingly valuable in forecasting the water supply of the succeeding season. In the field of land drainage, information concerning intensity and quantity of precipitation during storms is necessary in the design of drainage works. In flood-control projects, either on extensive water-sheds or in connection with localized storm-drain projects, rates of precipitation during periods ranging from 5 min. to several days in extent become necessary in the operation and design of such works.

Temperature.—Next in importance to precipitation, from the standpoint of the engineer, is air temperature. Information concerning this

[8] "The Technical Use That Engineers Make of U. S. Weather Observations," by Charles H. Lee, M. Am. Soc. C. E., Paper presented before the Am. Meteorological Soc., at Pasadena, Calif., June, 1931.

factor is necessary in financing irrigation projects in order to determine the length of the growing season, the character and variety of crops which can be grown successfully, and the number of crops which can be raised each year. Provision for ice control in reservoirs and canals becomes essential in regions where freezing temperatures prevail. Highway sub-grades must be protected from frost by means of adequate drainage. Artificial heating in buildings and temperature control in steam power plants depend upon a knowledge of air temperatures. Large outdoor construction operations are influenced by this phenomena, as are transportation schedules. Temperature is an essential factor in connection with evaporation from open-water surfaces, and oftimes must be the sole dependence in making evaporation studies where actual evaporation data do not exist.

Wind Movements.—Next to temperature in importance comes wind. The direction, velocity, and pressure of wind are important elements in the design and layout of high buildings, and oftimes in their location, particularly when odors, smoke, dust, or fumes exist. Wind velocity is an important factor in determining short-time variations in evaporation rates from open-water surfaces.

Evaporation.—This factor is of much importance in connection with water supplies in arid sections of the country where long periods occur without precipitation, and large losses result from open-water surfaces. In reservoirs and storage projects where hold-over storage lasting several years must be provided, exact knowledge of evaporation conditions becomes essential.

In addition to these four classes of meteorological data used extensively by engineers, such other data as relative humidity, barometric pressure, hours of sunlight, occurrence of killing frost, and current weather forecasts, are used quite extensively in engineering practice.

Intensive use of meteorological data in planning the construction and operation of private and public enterprises and improvements involving the expenditure of hundreds of millions of dollars has given rise to a feeling among those engineers who use such data that they could be collected and disseminated in a manner which would make them far more valuable to their users. Likewise, a growing feeling has developed among engineers that research in meteorology has not kept pace with experimentation in other fields of science which are utilized by the profession.

Attention has been called from time to time to specific instances where data collected and published by the Weather Bureau either lacked sufficient informational background or were collected in such a manner as to make their use misleading or the results obtained from such use unreliable. References to articles touching on such subjects will be found in Section V. As a result of considerable correspondence, the Administrative Board of American Engineering Council, at its meeting held May 12 and 13, 1930, authorized the appointment of a committee of three to "confer with officials of the Weather Bureau to determine the direction in which the Bureau might make its work more useful to engineers."

Following a rather insistent demand, particularly on the part of Western members of the Society, the same subject was presented to the Irrigation Division of the Society in the spring of 1930, and as an outgrowth of this, the present Special Committee on Meteorological Data was appointed by the Board of Direction at its meeting in April, 1931, "to give thought as to how the U. S. Weather Bureau could be made of greater service to engineers." The Committee of American Engineering Council was discontinued in the spring of 1931, in line with the policy of Council not to do anything which its member organizations are prepared and desire to do.

III.—Activities of the Committee

The Committee, with members in Los Angeles, and San Francisco, Calif., and in Washington, D. C., has carried on the bulk of its work by correspondence, the Los Angeles members being in close personal contact with each other. Several meetings of the four California members were held in Los Angeles. In the fall of 1931, a questionnaire was sent out to a list of 204 engineers throughout the country, three-fourths of these names being furnished by Presidents of Local Sections, the others being men who indicated to the Committee an interest in the work. Those replying to the questionnaire represented an excellent cross-section of the profession as to geographic location, types of employment, and specialized branches. Responses were received from consulting engineers, college professors, research workers, engineers engaged in power, irrigation, drainage, and flood-control projects, highway work, railroading, and municipal, county, State, and Federal work. Considerable material on the organization and functioning of the Weather Bureau was furnished the Committee by American Engineering Council.

Data relative to the distribution of existing Weather Bureau stations with respect to geography and altitude, and concerning the effect of elevation of recording instruments above the ground, were collected by a committee consisting of Kenneth H. Shaffer, *Chairman,* John Markley Server, Jr., and Arthur G. Pickett, Juniors, Am. Soc. C. E.; and a search, made in the library of the University of California, upon experiments carried on and published in *British Rainfall* fifty to sixty years ago was made by L. M. Snyder, Jun. Am. Soc. C. E. Various studies were made by members of the Committee individually and collectively, and the results of these are shown in Section V.

IV.—Replies to Questionnaire

From the 204 questionnaires mailed to members and others interested in the work, many excellent suggestions were received. Although expressing much criticism of the work of the Bureau, they likewise contained considerable favorable comment on certain portions of such work. In the questionnaire the question was asked as to whether the recipient would serve upon an advisory committee of one hundred to assist the main Committee in its work.

The following shows the total number of questionnaires sent out and the replies received:

Total questionnaires sent out	204
Number agreeing to serve upon advisory committee and submitting information and/or criticism	88
Number submitting suggestions or criticism, but not desiring to serve upon said committee	23
Total from whom information and/or criticism was received	111
Number replying, but furnishing little or no specific information or criticism	30
Total number of replies received	141

The replies showed an extensive use of the work and publications of the Weather Bureau by engineers, practically every type of data collected and published by the Bureau being utilized. In some cases one questionnaire would indicate the use of five or six types of data, and in other cases only one. An actual use more extensive than hereafter shown is probably made, as many replies were general in nature, stating that, "all data published" or "many of the data" were used. The following tabulation was taken from an analysis of replies received:

	No. using specific data
(1) Precipitation, used in connection with water supply studies	84
(2) Intensity of precipitation, in connection with flood-control and storm-drain projects	47
(3) Temperature, in connection with flood-control, irrigation, agriculture, heating and ventilating problems, construction projects, etc	33
(4) River gauge heights, in connection with flood control, construction projects, and navigation	30
(5) Snow conditions and water content of snow, in connection with water supply and flood control	20
(6) Evaporation, in connection with water supply and irrigation problems	15
(7) Weather forecasts and storm warnings, in connection with flood control, construction projects, etc	15
(8) Wind velocities and wind pressure, in connection with structural problems, evaporation studies, etc	14
(9) Flood warnings in connection with flood control	10
(10) Frost, in connection with construction operations, agricultural and irrigation work, highway design, etc	8
(11) Barometer, in connection with various phases of engineering work	6
(12) Other classes mentioned, as used by from one to three or four of those replying, included the use of all records in teaching, humidity, occurrence of lightning and thunder-storms, sea-water temperatures, and solar radiation.	

The Committee has carefully analyzed and segregated the replies and suggestions received to the questionnaires, and is at present engaged in making a study of them.

V.—Special Studies Made by the Committee

Appropriations.—Graphically, Fig. 1 shows the annual appropriation made for the support of the Weather Bureau for the fiscal years 1892 to 1931, inclusive. It will be noted that a quite uniform increase in appropriation prevailed until about 1921, when the amount appropriated increased con-

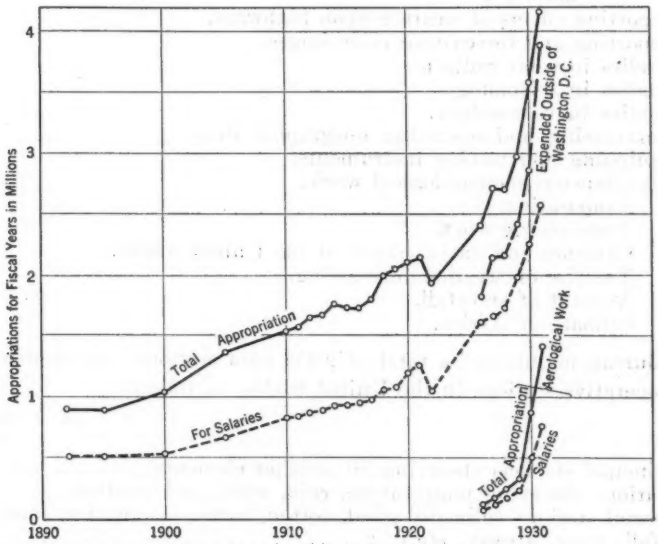

Fig. 1.—Appropriations Made for Support of U. S. Weather Bureau

siderably. This increase appears to be caused by the development of aerological work, which now absorbs one-third of the total appropriation, and also by salary increases resulting from a general survey of Government bureaus made by the Civil Service Commission about 1924. The ratio between salaries and general expense in the Bureau has been fairly constant throughout the period.

Organization.—The Act of Congress which created the Weather Bureau describes the duties of the Chief of the Bureau as follows:

"The Chief of the Weather Bureau, under the direction of the Secretary of Agriculture, shall have charge of forecasting the weather; the issue of storm warnings; the display of weather and flood signals for the benefit of agriculture, commerce, and navigation; the gauging and reporting of rivers; the maintenance and operation of sea coast telegraph lines, and the collection and transmission of marine intelligence for the benefit of commerce and navigation; the reporting of temperature and rainfall conditions for cotton interests; the display of frost, cold wave, and other signals; the distribution of meteorological information in the interest of agriculture and commerce, and the taking of such meteorological observations as may be necessary to establish and record the climatic conditions of the United States, or are essential for a proper execution of the foregoing duties."

The present activities of the Weather Bureau[9] cover the following subjects:

Weather reports and forecasting.
Climatological work.
Work in marine meteorology.
Work in agricultural meteorology.
Work in aerology.
Reporting effects of weather upon highways.
Reporting and forecasting river stages.
Studies in solar radiation.
Studies in seismology.
Studies in volcanology.
Maintaining and operating telegraphic lines.
Equipping and testing instruments.
Miscellaneous meteorological work:
- Evaporation.
- Atmospheric work.
- Cyclones and anti-cyclones of the United States.
- Treatise on weather forecasting.
- Amount of snowfall.
- Climate of Africa.

The Bureau maintains[10] a total of 2 358 paid stations, and approximately 4 500 co-operative stations in the United States, as follows:

	Number of stations
Principal stations observing all weather elements	206
Stations observing temperature, rain, wind, and weather	72
Special stations (climatological, cotton, cattle, sugar, rice, snowfall, river, airway, etc.)	2 080
Total paid stations	2 358
Co-operative stations (about)	4 500
Grand total	6 858

The five forecast districts have headquarters at Washington, D. C., Chicago, Ill., New Orleans, La., Denver, Colo., and San Francisco, Calif. Each State is arbitrarily made a climate unit for convenience in administration, for the collection and publication of climatological data, except that the New England States, Maryland, Delaware, Puerto Rico, and Hawaii each constitute a unit. There are forty-five of such units or districts. One principal station in each unit supervises the work of the district under the direction of a climatologist who has the title of Section Director. Great agricultural sections of the country are grouped according to the staple products therein, such as the cotton region, corn and wheat region, sugar and rice region, tobacco region, etc. Stations reporting river and flood information are grouped into districts chiefly by rivers and their immediately contiguous water-sheds, each with a central station within the district, although on large rivers the work is divided among several districts.

[9] "The Weather Bureau," Service Monographs of the U. S. Government, No. 9, Inst. of Governmental Research, N. Y., D. Appleton & Co., 1922, p. 16.
[10] Rept., Chf. of U. S. Weather Bureau, 1929–30, p. 41.

From the organization chart of the Bureau, Fig. 2, it is apparent that the organization structure rests on too broad a base, and undoubtedly a more efficient functioning could be obtained by grouping together as bureaus some of the present divisions which perform similar duties, leaving from three to five main divisions. No detailed study has been made of this subject, as this would require a much more intimate knowledge of the functioning of the Bureau than has been obtained by the Committee to date.

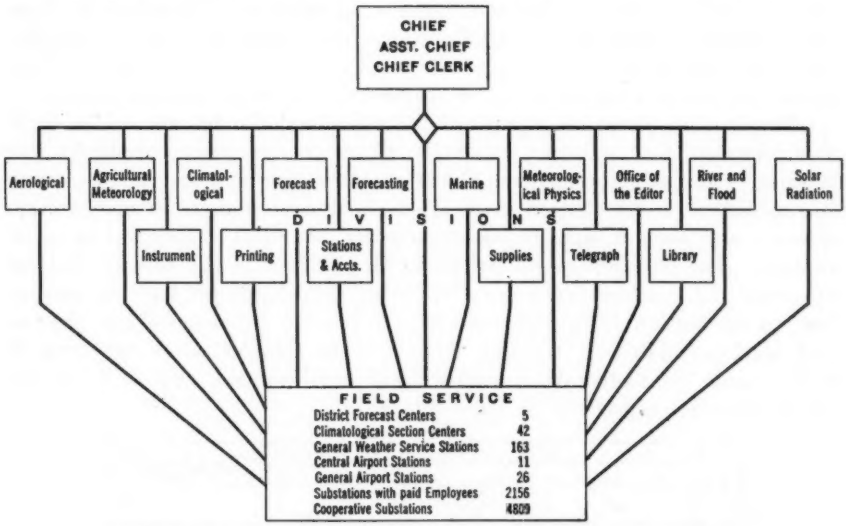

FIG. 2.—ORGANIZATION OF THE UNITED STATES WEATHER BUREAU

Personnel.—The technical personnel of the Bureau is under Civil Service regulation and is divided primarily into two groups, namely, the Professional Service and the Sub-Professional Service. The classification as to grades within each group is that followed throughout practically all the governmental departments employing help of this character. The Professional Service has eight grades, ranging in annual salaries from $2 000 to $9 000, and includes the technical employees of the Bureau. The Sub-Professional Service, with a maximum annual salary of $3 200, is made up primarily of skilled mechanics and assistants of lesser education and training.

Entrance requirements of the various grades as set out in the publications of the U. S. Civil Service Commission are fairly high. For example, the entrance qualifications for the grade of Junior Meteorologist, the lowest grade in the Professional Service, with a salary range from $2 000 to $2 600 per annum, call for training equivalent to graduation from a university or college of recognized standing, with major work in physics, and preferably including meteorology; knowledge of meteorology, astronomy, French or German, and mathematics, including geometry, trigonometry, and analytics; proficiency and ability in making rapid and accurate meteorological,

aerological, and solar radiation observations, reductions, and compilations for special study purposes; and accuracy, alertness, and adaptability. For the next higher grade, Assistant Meteorologist, with a salary range from $2 600 to $3 200 per annum, the qualifications, in addition to those required for a Junior Meteorologist, are either two years' post-gratuate study, or two years' successful experience in work of the grade and character required of a Junior Meteorologist. Those for Associate Meteorologist, with a salary range from $3 200 to $3 800 per annum, require, in addition to graduation, three years' study in meteorology, which study must have fulfilled the requirements for the degree of Doctor of Philosophy or Doctor of Science, or three years' professional experience of a responsible character in meteorology.

Entrance requirements for the Sub-Professional Service are not so high. Requirements for Assistant Observer, which is the fourth grade in this Service, with a salary range from $1 620 to $1 980 per annum, call for training equivalent to graduation from high school, with courses in meteorology, physics, and algebra, and at least two years' practical experience in meteorology. The entrance qualifications in Grade V, with the title of Aide or Observer and a salary range from $1 800 to $2 160, call for training equivalent to graduation from high school, with courses in meteorology, physics, and algebra, additional training equivalent to completion of one year of college work in meteorology, physics, and mathematics, and at least two years' practical experience in meteorology, etc.

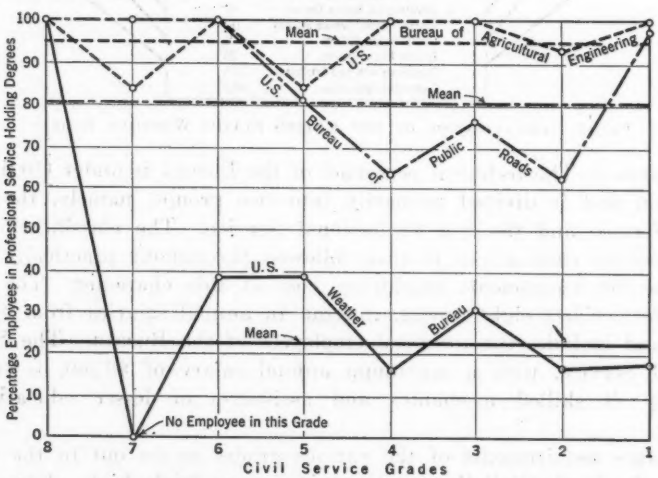

FIG. 3.—DEGREES HELD IN PROFESSIONAL SERVICE—COMPARISON OF THREE BUREAUS IN U. S. DEPARTMENT OF AGRICULTURE

An analysis of the technical personnel of the Weather Bureau in the Professional and Sub-Professional Services is given herewith in Table 1. Comparative studies of the personnel in the various grades of Professional Service in the Weather Bureau and two other governmental bureaus within the Department of Agriculture are likewise shown in graphical form in Fig. 3 which brings out some of the weaknesses of the personnel structure

TABLE 1.—TECHNICAL PERSONNEL OF U. S. WEATHER BUREAU: SALARIES, CIVIL SERVICE CLASSIFICATION, AND EDUCATION*

(Figures in bold-faced type, as **3**, are the number listed as having college degrees. Met.=Meteorologist; Obs.=Observer)

	PROFESSIONAL SERVICE									SUB-PROFESSIONAL SERVICE							Per cent of graduates	Percentage of technical personnel in classification
Grade	8	7	6	5	4	3	2	1		8	7	6	5	4				
Title	Chief	Assoc. Chief	Prin. Met.	Sr. Met.	Met.	Assoc. Met.	Asst. Met.	Jr. Met.	Sub-total	Chief Aide	Prin. Aide	Sr. Aide	Aide or Obs.	Asst. Obs.	Sub-total	Total		
Salary range (annual)	8 000 9 000	7 500 6 500	5 600 6 400	4 600 5 400	3 800 4 600	3 200 3 800	2 600 3 200	2 000 2 600		2 600 3 200	2 300 2 900	2 000 2 600	1 800 2 160	1 620 1 980				
WASHINGTON:																		
Administration	1 **1**	....	1 **1**	....	....	....	....	....	2 **2**	....	....	....	....	....	....	2 **2**	100.0	0.5
Investigation and service	....	....	6 **2**	5 **3**	....	5 **2**	9 **2**	2 **0**	27 **9**	3 **0**	....	....	....	....	3 **0**	30 **9**	30.0	6.9
Sub-totals	1 **1**	....	7 **3**	5 **3**	....	5 **2**	9 **2**	2 **0**	29 **11½**	3 **0**	....	....	....	....	3 **0**	32 **11**	34.4	7.4
FIELD:																		
Forecast districts	....	....	5 **2**	4 **1**	....	1 **1**	....	....	10 **4**	....	....	....	....	....	....	10 **4**	40.0	2.3
General weather service and research	....	....	2 **0**	6 **2**	1 **0**	19 **8**	79 **9**	75 **17**	182 **36**	....	....	8 **1**	2 **0**	....	10 **1**	192 **37**	19.13	44.0
Climatological section hydrographers	....	....	3 **2**	15 **6**	1 **0**	30 **9**	27 **10**	24 **4**	100 **31**	....	....	....	....	....	....	100 **31**	31.0	22.9
Frost service	....	....	....	1 **0**	....	....	2 **0**	4 **0**	7 **0**	....	....	1 **0**	2 **0**	....	3 **0**	10 **0**	0.0	2.3
Fire weather warning service†	....	....	....	....	....	....	5 **0**	1 **0**	6 **0**	....	....	....	1 **1**	....	1 **1**	7 **1**	14.3	1.6
Airways service†	....	....	1 **0**	....	4 **1**	9 **0**	14 **3**	32 **3**	60 **7**	....	....	5 **0**	7 **2**	7 **0**	19 **2**	79 **9**	11.4	18.1
Aerological investigation†	....	....	....	....	....	....	4 **0**	2 **1**	6 **1**	....	....	....	....	....	....	6 **1**	16.7	1.4
Sub-total	....	....	11 **4**	26 **9**	6 **1**	59 **18**	131 **22**	138 **25**	371 **79**	....	....	14 **1**	12 **3**	7 **0**	33 **4**	404 **83**	20.5	92.6
Totals	1 **1**	....	18 **7**	31 **12**	6 **1**	64 **20**	140 **24**	140 **25**	400 **90**	3 **0**	....	14 **1**	12 **3**	7 **0**	36 **4**	436 **94**	21.6	100.0
Percentage of graduates	100.0	....	38.9	38.7	16.7	31.3	17.1	17.8	22.5	0	....	7.1	25.0	0	11.1	21.6	....	.....
Percentage of personnel in classification	0.2	....	4.1	7.1	1.4	14.8	32.1	32.1	91.8	0.7	....	3.2	2.7	1.6	8.2	100.0	....	.....

* Data, except as to salary range and Civil Service grading, from *Miscellaneous Publications No. 123*, U. S. Dept. of Agriculture, "List of Technical Workers in the Department of Agriculture and Outline of Department Functions, July 1, 1931." Data on salary range, etc., from report of Director of the Budget, 1931.

† Net employees. There are other employees assigned to these classifications who are also listed in other classifications.

in the Weather Bureau. The apparently low percentage of employees in the Professional Service that hold collegiate degrees—only 22.5% of the total—is striking when compared with 80.3% for the Bureau of Public Roads and 95.0% for the Bureau of Agricultural Engineering.

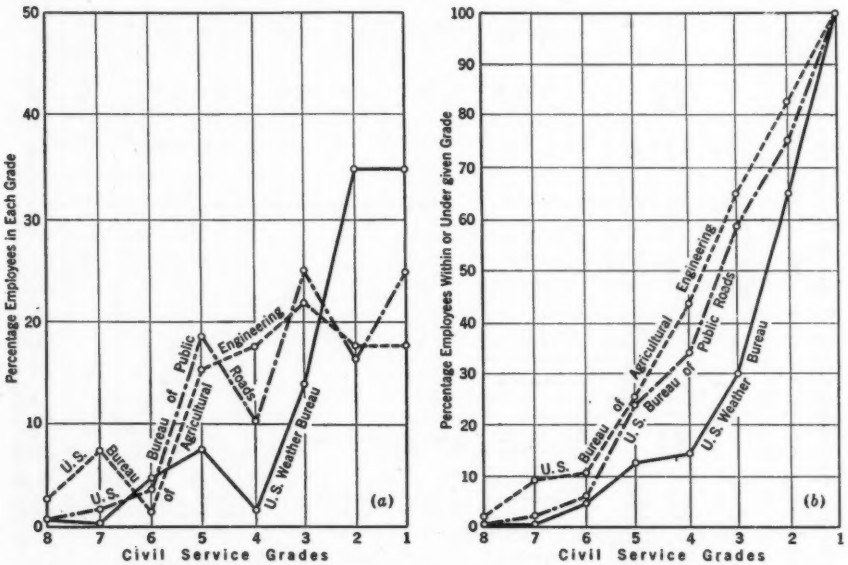

FIG. 4.—PERCENTAGE DISTRIBUTION OF EMPLOYEES IN PROFESSIONAL SERVICE—COMPARISON OF THREE BUREAUS IN U. S. DEPARTMENT OF AGRICULTURE

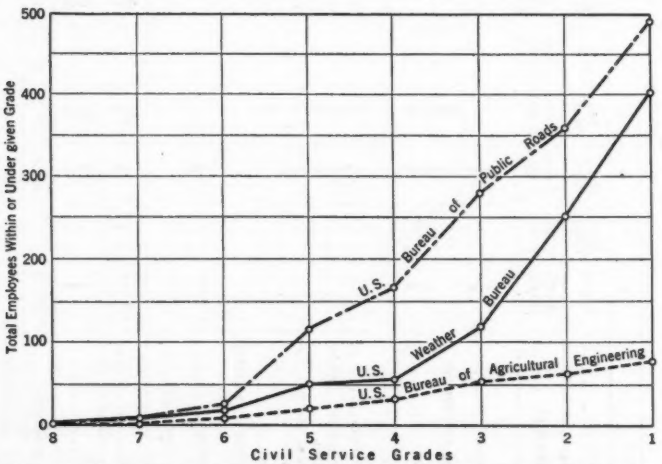
Total Employees Within or Under given Grade
Civil Service Grades
U.S. Bureau of Public Roads
U.S. Weather Bureau
U.S. Bureau of Agricultural Engineering

FIG. 5.—NUMERICAL DISTRIBUTION OF EMPLOYEES IN PROFESSIONAL SERVICE—COMPARISON OF THREE BUREAUS OF THE U. S. DEPARTMENT OF AGRICULTURE

The distribution of employees among the various grades in the Professional Service in the three Bureaus is likewise shown in Figs. 4 and 5.

These graphs also emphasize the lack of employees in the higher and more responsible grades in the Weather Bureau as compared with the other governmental bureaus. In Figs. 3, 4, and 5 the data, except as to salary range and Civil Service grading, were taken from *Miscellaneous Publications No. 123,* "List of Technical Workers in the Department of Agriculture and Outline of Department Functions, July 1, 1931"; and those as to salary range and Civil Service grading, from the Report of the Director of the Budget, 1931.

Salary ranges attaching to the various Civil Service grades within both the Professional and Sub-Professional Services appear to compare favorably

TABLE 2.—DISTRIBUTION OF PRECIPITATION STATIONS OF THE WEATHER BUREAU IN THE UNITED STATES BY STATES

(Number of Stations from Report of Chief of U. S. Weather Bureau, 1920-30)

State	Incorporated cities of more than 3 000 population	Area, in square miles	Number of stations	Square mile per station	Population, 1930	Stations per 100 000 population	Population per square mile
Alabama	41	51 998	74	703	2 646 248	2.8	50.9
Arizona	14	113 956	140	813	435 573	32.1	3.8
Arkansas	40	53 335	93	573	1 854 482	5.0	34.7
California	130	158 297	270	587	5 677 251	4.8	35.8
Colorado	24	103 948	121	860	1 035 791	11.7	10.0
Connecticut	28	4 956	15	330	1 606 903	0.9	324.0
Delaware	5	2 370	6	395	238 380	2.5	100.5
Florida	47	58 666	93	631	1 468 211	6.3	25.0
Georgia	60	59 265	64	926	2 908 506	2.2	49.1
Idaho	16	83 888	110	763	445 032	24.7	5.3
Illinois	167	56 665	97	583	7 630 654	1.3	134.7
Indiana	83	36 354	86	423	3 238 503	2.7	89.0
Iowa	70	56 147	121	464	2 470 939	4.9	44.0
Kansas	51	82 158	149	551	1 880 999	7.9	22.9
Kentucky	48	40 598	85	477	2 614 589	3.3	64.4
Louisiana	39	48 506	62	783	2 101 593	2.9	43.3
Maine	24	33 040	56	590	797 423	7.0	24.1
Maryland	18	12 327	51	242	1 631 526	3.1	132.4
Massachusetts	160	8 266	60	138	4 249 614	1.4	513.0
Michigan	104	57 980	154	376	4 842 325	3.2	83.5
Minnesota	53	84 682	84	1 007	2 563 953	3.3	30.3
Mississippi	32	46 865	64	733	2 009 821	3.2	42.8
Missouri	61	69 420	100	694	3 629 367	2.8	52.3
Montana	15	146 997	126	1 167	537 606	23.4	3.7
Nebraska	25	77 520	148	522	1 377 963	10.7	17.8
Nevada	5	110 690	50	2 210	91 058	54.8	0.8
New Hampshire	11	9 341	27	346	465 293	5.8	49.8
New Jersey	141	8 224	43	191	4 041 334	1.1	492.0
New Mexico	13	122 634	211	581	423 317	49.8	3.5
New York	175	49 204	117	421	12 588 066	0.9	256.0
North Carolina	54	52 426	105	499	3 170 276	3.3	60.4
North Dakota	12	70 837	100	708	680 845	14.7	9.6
Ohio	152	41 040	139	295	6 646 697	2.1	162.0
Oklahoma	56	70 057	100	701	2 396 040	4.2	34.2
Oregon	21	96 694	142	681	953 786	14.9	9.9
Pennsylvania	301	45 126	141	320	9 631 350	1.5	213.5
Rhode Island	6	1 248	4	312	687 497	0.6	551.0
South Carolina	31	30 989	56	553	1 738 765	3.2	56.1
South Dakota	11	77 615	96	808	692 849	13.9	8.9
Tennessee	43	42 022	79	532	2 616 556	3.0	62.2
Texas	135	265 896	252	1 054	5 824 715	4.3	21.9
Utah	16	84 990	131	648	507 847	25.8	6.0
Vermont	13	9 564	20	478	359 611	5.6	37.6
Virginia	37	42 627	55	775	2 421 851	2.3	56.8
Washington	30	69 127	197	351	1 563 396	12.6	22.6
West Virginia	34	24 170	93	260	1 729 205	5.4	71.5
Wisconsin	71	56 066	92	609	2 939 006	3.1	52.3
Wyoming	7	97 914	88	1 113	225 565	39.0	2.3
Total	2 730	3 026 789	4 767	635	122 775 046	3.9	40.6

TABLE 3.—Distribution of Precipitation Stations of the U. S. Weather Bureau According to Elevation*

State	Elevation of State†			Number of Stations Within Elevation Range													
	Maximum	Minimum	Mean	Below 0	0 500	500 1 000	1 000 1 500	1 500 2 000	2 000 3 000	3 000 4 000	4 000 5 000	5 000 6 000	6 000 8 000	8 000 10 000	10 000 12 000	12 000 plus	Total
Delaware	440	0	60	.....	7	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	7
Florida	325	0	100	.....	95	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	95
Louisiana	400	0	100	.....	56	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	56
District of Columbia	420	0	150	.....	3	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	3
Rhode Island	805	0	200	.....	5	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	5
New Jersey	1 805	0	250	.....	64	6	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	70
Mississippi	780	0	300	.....	67	22	9	1	.....	.....	.....	.....	.....	.....	.....	.....	99
Maryland	3 340	0	350	.....	45	12	1	1	3	.....	.....	.....	.....	.....	.....	.....	62
South Carolina	3 548	0	350	.....	43	30	2	.....	.....	.....	.....	.....	.....	.....	.....	.....	75
Massachusetts	3 505	0	500	.....	16	5	2	2	.....	.....	.....	.....	.....	.....	.....	.....	25
Alabama	2 407	0	500	.....	33	22	1	1	.....	.....	.....	.....	.....	.....	.....	.....	57
Connecticut	2 355	0	500	.....	8	4	1	.....	.....	.....	.....	.....	.....	.....	.....	.....	13
Georgia	4 768	0	600	.....	45	31	9	2	2	.....	.....	.....	.....	.....	.....	.....	89
Illinois	1 241	279	600	.....	25	92	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	117
Maine	5 267	0	600	.....	17	6	2	1	.....	.....	.....	.....	.....	.....	.....	.....	26
Arkansas	2 800	55	650	.....	57	12	11	3	1	.....	.....	1	.....	.....	.....	.....	85
Indiana	1 240	316	700	.....	16	78	4	.....	.....	.....	.....	.....	.....	.....	.....	.....	98
North Carolina	6 684	0	700	.....	44	17	2	4	7	4	.....	.....	.....	.....	.....	.....	78
Kentucky	4 150	257	750	.....	14	49	7	.....	.....	.....	.....	.....	.....	.....	.....	.....	70
Missouri	1 800	230	800	.....	14	98	29	1	.....	.....	.....	.....	.....	.....	.....	.....	142
Ohio	1 550	425	850	.....	.....	143	53	.....	.....	.....	.....	.....	.....	.....	.....	.....	196
Michigan	2 023	572	900	.....	.....	95	17	5	.....	.....	.....	.....	.....	.....	.....	.....	117
Tennessee	6 642	182	900	.....	20	40	14	9	2	.....	.....	.....	.....	.....	.....	.....	85
Virginia	5 719	0	950	.....	41	11	7	6	8	2	.....	.....	.....	.....	.....	.....	75
New Hampshire	6 288	0	1 000	.....	9	6	4	2	.....	.....	.....	.....	1	.....	.....	.....	22
Vermont	4 393	95	1 000	.....	4	15	1	.....	1	.....	.....	.....	.....	.....	.....	.....	21

* From *Bulletin W* of U. S. Weather Bureau. A few stations with elevations not given and a few with scattered records, are omitted from this tabulation.

† Elevation ranges of States, estimated by U. S. Geological Survey.

TABLE 3.—(*Continued*)

State	Elevation of State†			Number of Stations Within Elevation Range													
	Maximum	Minimum	Mean	Below 0	0 500	500 1 000	1 000 1 500	1 500 2 000	2 000 3 000	3 000 4 000	4 000 5 000	5 000 6 000	6 000 8 000	8 000 10 000	10 000 12 000	12 000 plus	Total
New York	5 344	0	1 000	.....	89	49	38	22	2	.....	.....	.....	.....	.....	.....	.....	200
Wisconsin	1 940	581	1 050	.....	.....	64	28	12	.....	.....	.....	.....	.....	.....	.....	.....	104
Iowa	1 675	477	1 100	.....	.....	65	91	3	.....	.....	.....	.....	.....	.....	.....	.....	159
Pennsylvania	3 213	0	1 100	.....	39	65	46	10	4	.....	.....	.....	.....	.....	.....	.....	164
Minnesota	1 920	602	1 200	.....	.....	38	69	1	.....	.....	.....	.....	.....	.....	.....	.....	108
Oklahoma	4 978	300	1 300	.....	4	38	36	12	5	4	1	.....	.....	.....	.....	.....	100
West Virginia	4 860	240	1 500	.....	2	41	13	9	12	2	1	.....	.....	.....	.....	.....	80
Washington	14 408	0	1 700	.....	68	26	22	25	35	3	.....	.....	.....	.....	.....	.....	179
Texas	8 700	0	1 700	.....	85	56	21	20	22	13	7	.....	.....	.....	.....	.....	224
North Dakota	3 468	790	1 900	.....	.....	17	22	40	20	.....	.....	.....	.....	.....	.....	.....	99
Kansas	4 135	700	2 000	.....	.....	24	51	15	30	12	.....	.....	.....	.....	.....	.....	132
South Dakota	7 242	962	2 200	.....	.....	.....	27	28	18	8	3	4	4	.....	.....	.....	92
Nebraska	5 300	825	2 600	.....	.....	3	25	24	29	20	3	.....	.....	.....	.....	.....	104
California	14 496	—276	2 900	8	142	43	27	37	48	34	27	16	9	3	.....	.....	394
Oregon	11 253	0	3 300	.....	55	16	22	20	25	23	31	6	1	.....	.....	.....	199
Montana	12 850	1 800	3 400	.....	.....	.....	.....	3	38	49	36	26	9	1	.....	.....	162
Arizona	12 611	100	4 100	.....	3	6	11	6	11	24	14	18	13	1	.....	.....	107
Idaho	12 655	720	5 000	.....	.....	2	4	4	36	38	36	25	9	.....	.....	.....	154
Nevada	13 145	470	5 500	.....	.....	.....	.....	1	2	7	33	23	34	.....	.....	.....	100
New Mexico	13 306	2 876	5 700	.....	.....	.....	.....	.....	.....	14	43	33	70	15	1	.....	176
Utah	13 498	2 000	6 100	.....	.....	.....	.....	.....	2	6	55	50	31	1	.....	.....	145
Wyoming	13 785	3 100	6 700	.....	.....	.....	.....	.....	.....	4	28	15	43	5	.....	.....	95
Colorado	14 420	3 350	6 800	.....	.....	.....	.....	.....	.....	9	28	25	56	56	10	1	185
Total	.....	.....	.....	8	1 235	1 347	729	330	363	276	346	242	280	82	11	1	5 250

* From *Bulletin W* of U. S. Weather Bureau. A few stations with elevations not given and a few with scattered records, are omitted from this tabulation.

† Elevation ranges of States, estimated by U. S. Geological Survey.

with salary ranges in other public technical organizations, and the proportion of employees in the respective grades in the Bureau of Public Roads and the Bureau of Agricultural Engineering appear quite reasonable. The personnel structure of the Weather Bureau, however, does not appear to be such as to attract into that service the type of men which its importance justifies, or to develop the spirit and morale which should exist within the ranks of an organization of this type.

Distribution of Stations.—Much criticism was found in replies to the questionnaire concerning the lack of uniformity both geographically and with respect to elevation above sea level, which exists concerning the distribution of principal and co-operative Weather Bureau stations. A study made of this and shown in Table 2 and Fig. 6, indicates a high density of stations in some regions and a low density in others.

A similar analysis with respect to elevation of stations in various States is given in Table 3. That relatively few stations are located in higher altitudes, wherein originate most of the water supplies of the country, is very apparent.

VI.—Exposure of Recording Instruments at Elevated Locations, and Changes in Location of Such Instruments

The most persistent criticism against current practices of the Weather Bureau which came to the attention of the Committee was directed against the practice of making instrumental observations at points considerably above the level of the surrounding locality, usually on the top of office buildings; also the practice of making frequent changes in the elevation or

TABLE 4.—Elevation of Instruments Above Surface of Ground at Principal U. S. Weather Bureau Stations
(Data from Report of Chief of Weather Bureau, 1929-30, p. 42.)

Elevation of instrument above ground surface, in feet	Rain gauges, number of instruments	Thermometers, number of instruments	Anemometers, number of instruments
0– 10	61	35	0
10– 20	0	27	0
20– 30	1	1	1
30– 40	9	3	1
40– 50	14	13	24
50– 60	18	13	39
60– 70	23	16	15
70– 80	17	20	12
80– 90	7	12	17
90–100	9	7	19
100–110	5	11	11
110–120	6	4	13
120–130	3	6	3
130–140	4	5	2
140–150	5	2	3
150–160	2	5	3
160–170	3	4	5
170–180	2	2	3
180–190	4	3	4
190–200	2	2	4
200+	12	14	27
Total	207	207	206

FIG. 6.—GEOGRAPHICAL DISTRIBUTION OF UNITED STATES WEATHER BUREAU STATIONS.

location of these instruments without noting such changes in any of the Weather Bureau publications, except in the Annual Report of the Chief of the Bureau.

In Table 4 is indicated the elevation above the ground surface of rain gauges, thermometers, and anemometers at the principal Weather Bureau stations, and from this it can be seen that many of them are located at elevations which cannot give correct ground conditions. Likewise, when recording instruments are changed in elevation, the records collected do not serve as a proper continuous index to meteorological conditions in the locality; and when these records are published without notations concerning such changes, a very erroneous impression is created. In Table 5 is shown changes in elevation of rain gauges, thermometers, and anemometers at the principal stations of the Bureau during the nine years from 1922 to 1929, inclusive. The only place where such changes have been noted has been in the Annual Report of the Chief of the Bureau for the then current year. Because of the personal experiences of members of the Committee, as well as the many criticisms expressed in the questionnaires concerning the effect of changes in elevation and location upon resulting records, the Committee has made a more or less intensive investigation of this subject.

Precipitation.—The fact that a rain gauge exposed at an elevation above the ground surface receives a catch of rain less than one exposed on the ground was determined by experiments carried on by Dr. Heberden about 1766.[11] Three rain gauges were established, one in the garden at Westminster, one on a house in the neighborhood, and a third on the square tower at Westminster Abbey. The results for the year 1766 were as follows:

Location of gauge	Total rain	Ratio
Garden (height about 1 ft.).....	22.61 in.	100%
House top (height about 30 ft.).....	18.14 in.	80%
Abbey Tower (height about 151 ft.)..	12.10 in.	54%

During the years 1832 to 1835 Messrs. Phillips and Gray carried on similar experiments at York, England, with similar results. As a conclusion from these experiments, Professor Phillips developed the fact that diminution of amount of rain obtained at an elevated gauge varied as the square root of the height above the ground.

Considerable experimentation took place in England on the subject from just before 1870 until 1880, and the results were published in issues of *British Rainfall* during that period. The experimenters reached the same conclusions as those arrived at by Messrs. Phillips and Gray, although other factors, such as position of the gauge upon the elevated structure, intensity of precipitation, direction and velocity of wind, etc., appeared to enter into the relation. In a pamphlet recently published by the India Meteorological Department[12] the results of these experiments, as well as of other observa-

[11] "On the Rainfall Observations Made at York Minster," by John Phillips and G. J. Symons, *British Rainfall*, 1871, p. 41.

[12] "Historical Note on Catch of Rain Gauges," by H. R. Puri, Scientific Notes, Vol. 3, No. 23, India Meteorological Dept., 1931, p. 44.

TABLE 5.—Changes in Elevation of Recording Instruments at Principal United States Weather Bureau Stations, 1922 to 1929, Inclusive
(From Annual Reports of Chief of the Weather Bureau)

Year	Station	Date of change	Elevation of Thermometer Above Ground, in Feet			Elevation of Rain Gauge Above Ground, in Feet			Elevation of Anemometer Above Ground, in Feet		
			From	To	Difference	From	To	Difference	From	To	Difference
	Hatteras	Mar. 8	12	11	−1	.....	.....	.....	.....	.....	.....
1922	Ithaca	June 9	82	5	−77	.....	.....	.....	.....	.....	.....
	Santa Fe	Mar. 29	57	38	−19	49	31	−18	66	53	−13
	Tampa	Dec. 11	.....	.....	.....	.....	.....	.....	92	87	−5
	Dayton	July 3	181	137	−44	176	132	−44	216	173	−43
	Duluth	Oct. 20	11	5	−6	.....	.....	.....	.....	.....	.....
1923	Independence	June 30	9	5	−4	.....	.....	.....	41	25	−16
	Sandusky	Jan. 1	.....	.....	.....	.....	.....	.....	103	70	−33
	Tacoma	May 15	113	172	+52	104	165	+61	120	201	+81
	Green Bay	Mar. 3	.....	.....	.....	.....	.....	.....	144	141	−3
	Louisville	June 29	219	188	−31	214	183	−31	255	234	−21
1924	Phœnix	Sept. 4	11	10	−1	68	56	−12	81	82	+1
	Terre Haute	May 16	.....	.....	.....	84	86	+2	.....	.....	.....
	Trenton	July 18	.....	.....	.....	152	105	−47	.....	.....	.....
	El Paso	July 1	110	152	+42	102	145	+43	133	175	+42
1925	Greenville	Oct. 1	113	139	+26	102	131	+29	122	146	+24
	Baltimore	July 21	.....	.....	.....	.....	.....	.....	113	215	+102
	Corpus Christi	Nov. 19	.....	.....	.....	.....	.....	.....	73	78	+5
	Dallas	Oct. 1	109	220	+111	92	194	+102	117	227	+110
1926	Evansville	July 1	139	76	−63	135	74	−61	175	116	−59
	Houston	Mar. 1	111	292	+181	104	286	+182	121	314	+193
	Meridian	Apr. 28	85	87	+2	62	64	+2	93	95	+2
	Roseburg	Sept. 1	9	27	+18	4	26	+22	57	60	+3
	Chattanooga	July 5	189	190	+1	.....	.....	.....	213	215	+2
	Lincoln	Oct. 11	.....	.....	.....	.....	.....	.....	84	81	−3
1927	Miami	July 9	.....	.....	.....	.....	.....	.....	79	164	+85
	Milwaukee	Apr. 19	.....	.....	.....	.....	.....	.....	139	221	+82
	Roseburg	July 1	27	75	+48	26	60	+34	60	99	+39
	Sandusky	Mar. 13	62	5	−57	54	3	−51	70	67	−3
	Albany	Oct. 12	102	107	+5	.....	.....	.....	.....	.....	.....
	Columbia, Mo	Mar. 22	11	6	−5	.....	.....	.....	.....	.....	.....
	Independence	July 1	5	6	+1	.....	.....	.....	25	27	+2
	Lansing	Feb. 21	11	6	−5	.....	.....	.....	62	49	−13
	Little Rock	Mar. 12	.....	.....	.....	.....	.....	.....	144	153	+9
1928	Miles City	Jan. 11	.....	.....	.....	39	41	+2	.....	.....	.....
	Philadelphia	Oct. 27	.....	.....	.....	.....	.....	.....	182	367	+185
	Red Bluff	July 1	50	5	−45	40	3	−37	56	58	+2
	Springfield, Ill	Nov. 1	10	154	+144	4	148	+144	91	191	+100
	Springfield, Mo	Oct. 1	97	65	−32	65	57	−8	113	79	−34
	Wassau	Sept. 27	.....	.....	.....	3	4	+1	.....	.....	.....
	Baker	Nov. 30	.....	.....	.....	40	44	+4	.....	.....	.....
	Boston	June 7	115	106	−9	154	96	−58	188	165	−23
	Brownsville	Feb. 1	53	83	+30	45	74	+29	61	100	+39
	Cleveland	June 27	190	267	+77	165	264	+99	201	337	+136
1929	Des Moines	Oct. 1	84	5	+79	76	3	+73	97	99	+2
	Hatteras	Feb. 7	11	5	−6	.....	.....	.....	.....	.....	.....
	Medford	Nov. 1	4	29	+25	67	25	−42	80	58	−22
	Miami	July 1	71	124	+53	64	117	+53	164	168	+4
	Phœnix	Jan. 2	.....	.....	.....	56	81	+25	82	107	+25
Total Changes in Elevation:		.....	.....	.....	33	.....	.....	27	.....	.....	38
Plus		.....	.....	.....	17	.....	.....	17	.....	.....	24
Minus		.....	.....	.....	16	.....	.....	10	.....	.....	14
Maximum Changes in Elevation:											
Plus — ft		.....	.....	.....	181	.....	.....	182	.....	.....	193
Minus — ft		.....	.....	.....	−77	.....	.....	−61	.....	.....	59
Extent of Change:											
0 to 50 ft	No.+	.....	.....	.....	10	.....	.....	10	.....	.....	15
	No.—	.....	.....	.....	13	.....	.....	8	.....	.....	13
50 to 100 ft	No.+	.....	.....	.....	4	.....	.....	4	.....	.....	3
	No.—	.....	.....	.....	3	.....	.....	2	.....	.....	1
100 to 150 ft	No.+	.....	.....	.....	2	.....	.....	1	.....	.....	4
	No.—	.....	.....	.....	0	.....	.....	0	.....	.....	0
150 to 200 ft	No.+	.....	.....	.....	1	.....	.....	2	.....	.....	2
	No.—	.....	.....	.....	0	.....	.....	0	.....	.....	0

Note.— No changes noted in elevation of barometers.

tions throughout the world, have been published. Table 6 is taken from this pamphlet and the results are shown graphically in Fig. 7, which indicates that the deficiency follows the parabolic law. In 1888, Abbé,[13] discussing

TABLE 6.—ALTITUDE AND RELATIVE CATCH OF RAIN

Situation and size of gauge	Years of record	Altitude, in meters	Percentage of relative catch
Calne, 5 in. and 8 in.	4	0	100
		1	90
Castleton, 5 in. and 8 in.	3	2	88
		3	86
Rotherham, 5 in.	8	4	85
		5	85
St. Petersburg: Central Physical Observatory, 10 in.	10	6	84
London: Westminster Abbey	1	9.1	77
Emden	2	11	72
St. Petersburg: Central Physical Observatory	1	13	68
York: Museum	3	13	80
Calcutta: Alipore Observatory	7	15	87
Woodside: Walton-on-Thames	1	15	73
Philadelphia: Frankford Arsenal	3	16	95
Sheerness: Water-works	3	21	52
Whitehaven: St. James Church	10	24	66
St. Petersburg: Central Physical Observatory	10	25	59
Paris: Astronomical Observatory	40	27	81
Dublin: Monkstown	6	27	64
Oxford: Radcliffe Observatory	8	34	59
Copenhagen: Observatory	4	36	67
London: Westminster Abbey	1	46	52
Chester: Leadwords	2	49	61
Wolverhampton: Water-works	3	55	69
York: Minster	3	65	60
Boston: St. Botolph's Church	2	79	47

the various investigations which had been made on this subject, proposed a formula, as follows:

$$D = K\sqrt{A}$$

in which, D = deficiency of catch, expressed in percentage of total catch; A = altitude of gauge, in meters; and K = a constant, given by Abbé as 0.06, but stated to vary with the geographical location of the gauge and with rainfall conditions.

Fanning[14] makes reference to differences observed in catch at Greenwich Observatory. A 50-year record of precipitation from 1865 to 1914, inclusive, shows averages, as follows:

Location of gauge	Average precipitation	Ratio
On ground	24.25 in.	100%
At 22 ft.	21.28 in.	88%
At 50 ft.	15.38 in.	63%

The late Desmond FitzGerald,[15] Past-President and Hon. M. Am. Soc. C. E., in discussing observations of maximum rates of rainfall at Chestnut Hill Reservoir, Boston, Mass., between the years 1879 and 1889, stated, "it is already a well-established fact that the higher we rise above the ground the

[13] *Bulletin 7*, Appendix 1, Forestry Dept., U. S. Dept. of Agriculture.

[14] "Practical Treatise on Hydraulics and Water Supply," by John T. Fanning, M. Am. Soc. C. E., 1895, p. 64.

[15] "Maximum Rates of Rainfall," *Transactions*, Am. Soc. C. E., Vol. XXI (1889), p. 94.

less rain we can collect, unless special construction is resorted to." In this paper corrections were applied to records obtained at an elevation above the ground surface to make them comparable with ground-level records.

Considerable discussion took place in the *Transactions* of the Society in 1908[16] pertaining to the San Francisco precipitation record as published

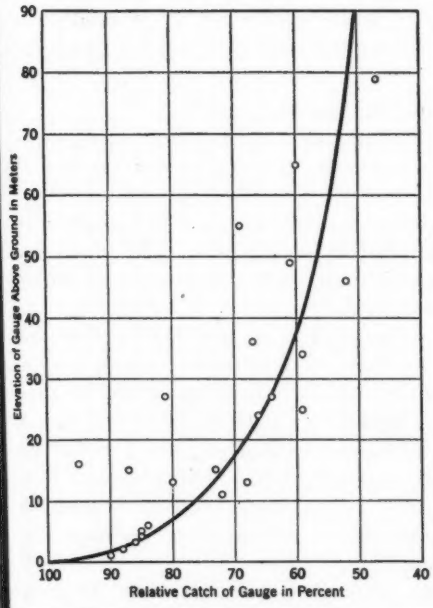

FIG. 7.—RELATION OF CATCH OF RAIN GAUGES TO ELEVATION ABOVE GROUND LEVEL.

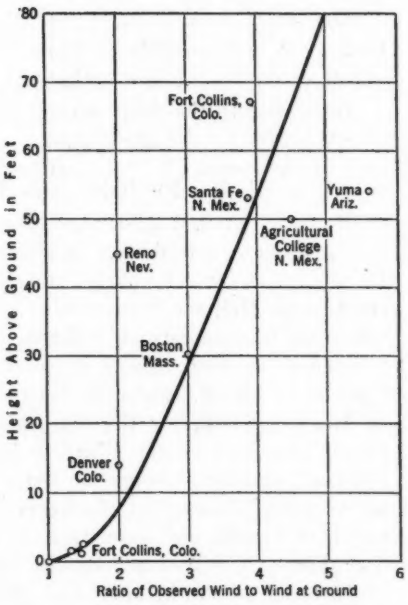

FIG. 8.—WIND VELOCITY REDUCTION AS GROUND LEVEL IS APPROACHED.

by the Weather Bureau. The published Weather Bureau record commences in 1849-50, and the recording instruments have been exposed at varying elevations above the ground. In the earlier years, as stated in one of the discussions of Mr. Grunsky's paper, the published record is a composite of a number of records kept at various sections of the city; no information is given, however, as to the location of these various records. The published Weather Bureau record is compared in the discussion with a record kept from 1865-66 to 1900-01 by a Mr. John Pettee, "at a single gauge, near the ground, under unchanging conditions" (page 527), although this point is disputed in another discussion (page 537). The results as published, however, show that the mean of the Pettee record for the period 1865-66 to 1891-92 was 97.2% of the mean of the Weather Bureau record for that period; while for the period, 1892-93 to 1900-01, the mean of the Pettee record was 134.4% of the published Weather Bureau record. During the latter period the Weather Bureau gauge was located on top of the Mills Building. The Committee is investigating this matter further.

[16] "Rainfall and Run-Off near San Francisco, California," by C. E. Grunsky, Past-President, Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXI (1908), p. 496.

In "Elementary Meteorology" [17] William Moran Davis states,

"Records of rain and snow from gauges on buildings in cities are, as a rule, defective, because of the eddies of wind by which too much or too little rain is carried into the gauge. Such measures may serve to indicate generally whether the fall is light or heavy, as is required in weather reports, but they should not be accepted for the climatic tables for the district."

Dr. Willis I. Milham, states,[18] "A rain gauge should not be exposed on a roof, as it is impossible to know what effect the eddies caused by buildings will have on the amount collected."

Referring to the Abbé formula, Willis L. Moore states,[19]

"As to the height of the gauge above ground, there is a very rapid diminution of the catch. * * * In perfect calms this error does not exist, and in strong winds with light rains it is much larger than the above rule."

Professor Alfred J. Henry[20] recognizes that rain-gauge roof-top records are inaccurate, but doubts whether satisfactory corrections could be made. In the review of a 3-volume treatise on rainfall on German water-sheds by Dr. George Hellman,[21] attention is drawn to the variation in recording rainfall from instruments at different elevations. It is stated therein that the differences between gauges at ground level and at 1 m. above the ground is 6 per cent. B. R. Laskowski,[22] commenting upon observations made by him at Topeka, Kans., for the five seasons, 1924-25 to 1928-29, inclusive, using records obtained by the Weather Bureau gauge, located on top of a 6-story building, and by a gauge located on the ground at his residence seventeen blocks away, shows that the average precipitation from April to September was 11% higher, and from October to March, 6% higher at the ground station than at the Weather Bureau station.

In order to obtain first-hand information on this subject at Los Angeles, the Committee arranged, through the courtesy of Carl B. Wirsching, M. Am. Soc. C. E., President of the Board of Public Works, J. J. Jessup, M. Am. Soc. C. E., City Engineer, and Leroy W. Armstrong, M. Am. Soc. C. E., Storm-Drain Engineer, for the installation of three gauges at varying elevations at the City Hall. These gauges were installed during the spring of 1932, and it is proposed to continue the observations over an extended period. The results to date (June, 1932), together with records of the Weather Bureau gauge, located on the roof of an 11-story building six blocks south of the City Hall, are given in Table 7.

Wind.—Wind velocity increases very considerably with elevation above the ground surface. Milham[23] states,

"Buildings increase the wind velocity and also make the wind gusty. * * * Wind velocity increases markedly with altitude. The increase is very rapid in the first 100 or 200 feet, particularly over land."

"On top of the Eiffel Tower, at an altitude of 990 feet, the average velocity is 3.1 times the velocity at an altitude of 60 feet."

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[17] See p. 290.
[18] "Meteorology," by Willis I. Milham, p. 245.
[19] "Descriptive Meteorology," by Willis L. Moore, p. 210.
[20] *Monthly Weather Review*, November, 1901, p. 500.
[21] *Loc. cit.*, July, 1907, p. 306.
[22] *Loc. cit.*, December, 1929, p. 506.
[23] "Meteorology," by Willis I. Milham, p. 65.

TABLE 7.—Comparison of Catch of Rain, in Inches, in United States Weather Bureau Gauge, and Gauges at Various Elevations Above Ground at City Hall, Los Angeles, California, for 1932

Date, 1932	City Hall Gauges: Distance Above Ground at Lawn Gauge,			Weather Bureau Gauge:¶ Distance above ground at Sixth and Main Streets, 151 ft.	Average wind velocity from Weather Bureau gauge, in miles per hour
	Lawn,† 3 ft.	Roof,‡ 73 ft.	Tower,§ 459 ft.		
February 10...	0.22	0.22	0.05	0.0	5.4
February 15–16	1.48	1.39	0.72	1.24	8.1
February 17*..	0.40	0.38	.....	T	6.6
February 18–19	0.27	0.27	0.16	0.23	5.3
April 25–26....	0.31	0.31	0.20	0.25	6.9
June 3–4–5.....	0.13	0.14	0.12	0.11	...
Totals.....	2.41	2.33	1.25	1.83	...
Ratio.....	100.0	96.6	51.8	76.0	...

* Not included in totals. Tower record lost for this day.
† Standard gauge set on lawn to south of City Hall, top of gauge 3 ft. above ground. Exposure excellent from all sides.
‡ Standard gauge set on roof of south wing of City Hall, 35 ft. north of south parapet and 20 ft. east of west parapet. Exposure fair. Tower to northwest of gauge.
§ Standard gauge lashed to railing of platform on top of tower of City Hall, 459 ft. above ground at lawn gauge, until February 17; then set out from platform on bracket. Exposure excellent.
¶ On roof of 11-story Central Building, 151 ft. above ground at this location, which is six blocks south of City Hall. Exposure very good for a roof-top gauge.

E. S. Nichols[24] points out that when higher buildings are built around a Weather Bureau station, very important changes take place in the direction of the wind and in the wind velocity. Using New York City as an example between 1901 and 1908, his paper shows that the number of days per year having a velocity of 40 miles per hour declined from 90 in 1900, to 47 in 1906; to 37 in 1907; to 44 in 1908; and to 49 in 1909; whereas at Block Island the number of days remained fairly constant.

Dr. Hellman,[25] commenting upon the records of three anemometer stations observed in Berlin, Germany, for the five years, 1911 to 1915, inclusive, found very eccentric variations. The stations were in three different sections of the city. He draws conclusions that, even in a relatively clear space, wind records are affected by buildings, and that obstructions render city readings almost worthless.

William Charles Devereaux[26] notes marked differences in wind velocity and direction at the Government station in Cincinnati, Ohio, and the observatory at Clifton, with anemometers 150 and 250 ft., respectively, above the ground. During April, 1915, a south wind was recorded at the Government station only 7.4% of the time, with an average velocity of 2.1 miles per hour; while at the Observatory such wind occurred 16.1% of the time, with an average velocity of 5.5 miles per hour. He states that during two years of record the south wind at the Government Building was deficient 67% in duration and 60% in velocity, as compared with observations at the Observatory.

[24] *Monthly Weather Bureau*, October, 1910, p. 471.
[25] *Loc. cit.*, November, 1920, p. 637.
[26] *Loc. cit.*, May, 1917, p. 224.

In Fig. 8 a curve is shown[27] based on observations taken at various points of the country concerning the relationship between wind velocities observed above the ground surface with velocities at the ground surface.

Temperature.—Temperature records are likewise affected by elevation above the ground surface, by surrounding buildings, and particularly by air currents and heated air rising from the top of steam-heated buildings. Milham[28] states,

"In a city, since air stagnates in a narrow street, the best location [for a thermometer] is the roof of some high building. This does not, of course, give the temperature of the streets, where people must live, but it probably does give a close approximation to the real air temperature of the country immediately surrounding the city."

Differences in monthly mean temperatures at the Weather Bureau station at St. Louis, Mo.,[29] where instruments are exposed on a building 110 ft. above the ground, and in Forest Park near-by, where instruments are located 10 ft. above the ground, ranged from 1.5° to 9°, with an average of 4.6°, the Weather Bureau instruments always being higher.

Messrs. C. H. McLeod and H. T. Barnes,[30] discussing temperature differences between instruments exposed at Montreal, Que., Canada, and at Mount Royal, at a distance horizontally of 3 000 ft. apart, the latter being 613 ft. higher in elevation, found that the lower station was always cooler, ranging from 3.7 to 11.6 degrees.

Citing records obtained on top of the Post Office Building at Vicksburg, Miss., 63 ft. above the ground, and at two stations in the lowlands of the valley near the city, with instruments 4 ft. above the ground, W. S. Belden[31] found through experiments carried on for four months, that on fifteen mornings the temperature at the lower stations was 10° lower than that found on top of the Post Office. The averages for March and April, 1907, were 3.4° and 3.3° lower at the ground stations than at the Weather Bureau station.

Professor J. Warren Smith,[32] citing temperature and precipitation records taken at the campus of Ohio State University and in the City of Columbus, Ohio, where instruments were located 6 ft. and 170 ft. above the ground, respectively, and observations were carried on for twenty-nine years, found that the average annual temperature at the Weather Bureau station in the City of Columbus was 1.2° lower, the mean maximum temperature was 0.3° higher, and the mean minimum temperature, 3.3° lower.

Edward B. Coberly[33] quotes information received from Mr. H. Myer concerning the minimum temperature for New Orleans, La., on November 16, 1901, which was recorded by the Weather Bureau as 44°, while at Mr.

[27] "Evaporation from Free Water," by Carl Rohwer, Assoc. M. Am. Soc. C. E., *Technical Bulletin 271*, U. S. Dept. of Agriculture in Co-Operation with Colorado Agri. Experiment Station, 1931, p. 75.

[28] "Meteorology," p. 58.

[29] *Monthly Weather Review*, January, 1902, p. 12.

[30] *Loc. cit.*, November, 1906.

[31] *Loc. cit.*, May, 1907, p. 219.

[32] *Loc. cit.*, January, 1912, p. 30.

[33] *Loc. cit.*, April, 1912, p. 573.

Myer's residence, twenty-five miles south of New Orleans, a temperature of 32° was recorded, with ice forming as thick as window glass.

Floyd D. Young,[34] Senior Meteorologist of the Weather Bureau, shows differences ranging from 25° to 30° occurring on a hillside with a difference

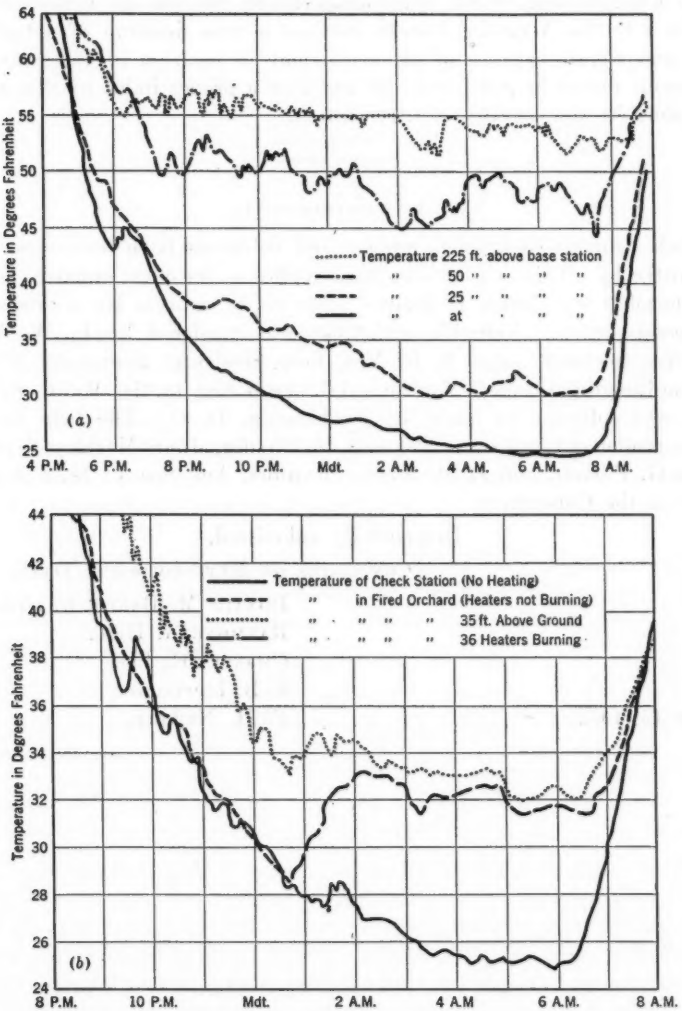

FIG. 9.—RELATION OF TEMPERATURE AND ELEVATION ABOVE GROUND.

in elevation of 225 ft. In the same *Bulletin*[35] a difference of 5° in temperature is shown between thermometers exposed near the ground and those exposed 35 ft. above the ground. These observations are shown in graphical

[34] "Frost and Prevention of Frost Damage," by Floyd D. Young, *Farmers' Bulletin 1588*, U. S. Dept. of Agriculture, April, 1929, p. 3.

[35] *Loc. cit.*, p. 17.

form in Fig. 9. It will be noted in Fig. 9(*b*) that orchard heaters were lighted between midnight and 1:00 A. M., which prevented the temperature, 35 ft. above the ground, from following the temperature of the other station.

Numerous other citations from meteorological literature might be given where precipitation, wind, and temperature records as collected and/or published by the Weather Bureau are not a true measure of actual conditions, and where, because of the movement in location of instruments, the continuous record as published does not give a proper index of climatological conditions in the locality of the station.

ACKNOWLEDGMENTS

Much valuable assistance, advice, and criticism have been rendered the Committee by all those who submitted replies to its questionnaire, and since their number was large, it desires herewith to express its appreciation of such co-operation. Valuable assistance was rendered by L. W. Wallace, Executive Secretary, and B. R. Van Leer, Assistant Secretary, of American Engineering Council, for material pertaining to the Weather Bureau, which was collected by them in Washington, D. C. The help in special studies rendered by Messrs. Kenneth H. Shaffer, John Markley Server, Jr., Arthur G. Pickett, and L. M. Snyder, Juniors, Am. Soc. C. E., was of great service to the Committee.

Respectfully submitted,

COMMITTEE ON METEOROLOGICAL DATA,

DONALD M. BAKER, *Chairman*,
RAYMOND A. HILL,
CHARLES H. LEE,
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F. H. NEWELL.

June 15, 1932.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

EFFECT OF BENDING WIRE ROPE

Discussion

BY MESSRS. A. S. RAIRDEN, AND F. C. CARSTARPHEN

A. S. RAIRDEN,[34] ESQ. (by letter).[34a]—The author has contributed some valuable information on a subject that has been under controversy for considerable time. As pointed out, bending stress formulas on wire rope developed, respectively, by Reuleux, Rankine, and Chapman give bending stress values out of proportion to the breaking strength of the rope. The problem is solved most easily and accurately by the development of empirical formulas by extending breaking-strength tests of wire rope bent to varying radii of curvature.

A series of tests were conducted to determine the breaking strength of $\frac{5}{8}$-in. rope (improved plow steel), of varying constructions bent over sheaves, as given in Table 11. The sheaves contained **U**-grooves conforming to the contour of the $\frac{5}{8}$-in. ropes.

TABLE 11

Diameter, in inches	3	6	8	10	16
Ratio of sheave diameter to rope diameter	4.8	9.6	12.8	16.0	25.6

For each test a length of wire rope was carefully socketed on both ends. The wire-rope sample was bent around the sheave (which was free to revolve around its axis), and anchored to the stationary head of a 250 000-lb. vertical testing machine (see Fig. 13). The two ends of rope, leading vertically downward and parallel to each other, were securely attached by sockets to a horizontal bar which, in turn, was attached to the movable head of the testing machine. Gradually the load was applied to the specimen bent around the sheaves until complete rupture occurred.

NOTE.—The paper by Frederick C. Carstarphen, M. Am. Soc. C. E., was published in December, 1931, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: April, 1932, by Messrs. Robert C. Strachan, and C. D. Meals; and May, 1932, by Messrs O. P. Erickson, F. W. Deck, and O. G. Julian and J. C. Damon.

[34] Chf. Engr., Am. Cable Co., Inc., New York, N. Y.

[34a] Received by the Secretary November 11, 1932.

Corresponding to each series of bending tests, a regular straight breaking-strength test of the rope was also conducted. In this test the straight sample of rope was subjected to loads of gradually increasing intensity until it broke.

The load applied to the bent specimen at rupture was divided by two, in order to obtain the load on one end. The value thus obtained was subtracted

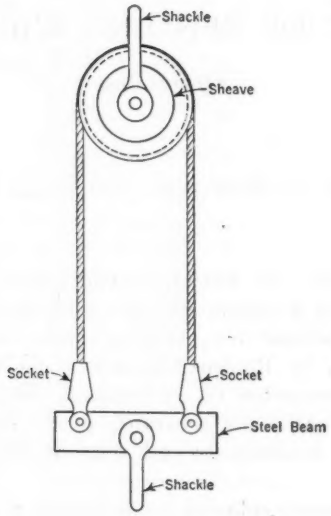


FIG. 13.

from the breaking strength of the wire rope obtained by a straight pull, in order to determine the loss in breaking strength of the rope due to being bent around the sheave. In each case this loss is expressed in terms of percentage loss of the breaking-strength value of the rope when pulled to destruction straight. The breaking-strength losses corresponding to each sheave diameter factor are interpreted graphically in Fig. 14, which also shows the cross-sections of the corresponding ropes. The details of construction of these ropes are listed in Table 12.

The important difference between the sample in Fig. 14(*b*) (regular lay rope) and that in Fig. 14(*c*) (lang lay rope) may be studied in Fig. 15. The details of construction are absolutely identical in all respects except that the direction of twist in the case of the lang lay rope (Fig. 15(*b*)) is the same for the wires in the strands and the strands in the rope. In the case of the regular lay rope the direction of twist of the wires in the strands is opposite to that of the strands in the rope.

The length of lay or pitch of the wires in the strands and the strands in the rope are, in all cases, of such a proportion that the imaginary angles formed by a wire with respect to the strand axis or by a strand with respect to the rope axis are equal and equivalent to 18.5° in magnitude.

In the case of a regular lay rope the angle of lay of the wires is equal and opposite in value to that of the lay of the strands. The resultant angle, therefore, is zero, and the wires on the crowns of the strands run parallel to the rope axis.

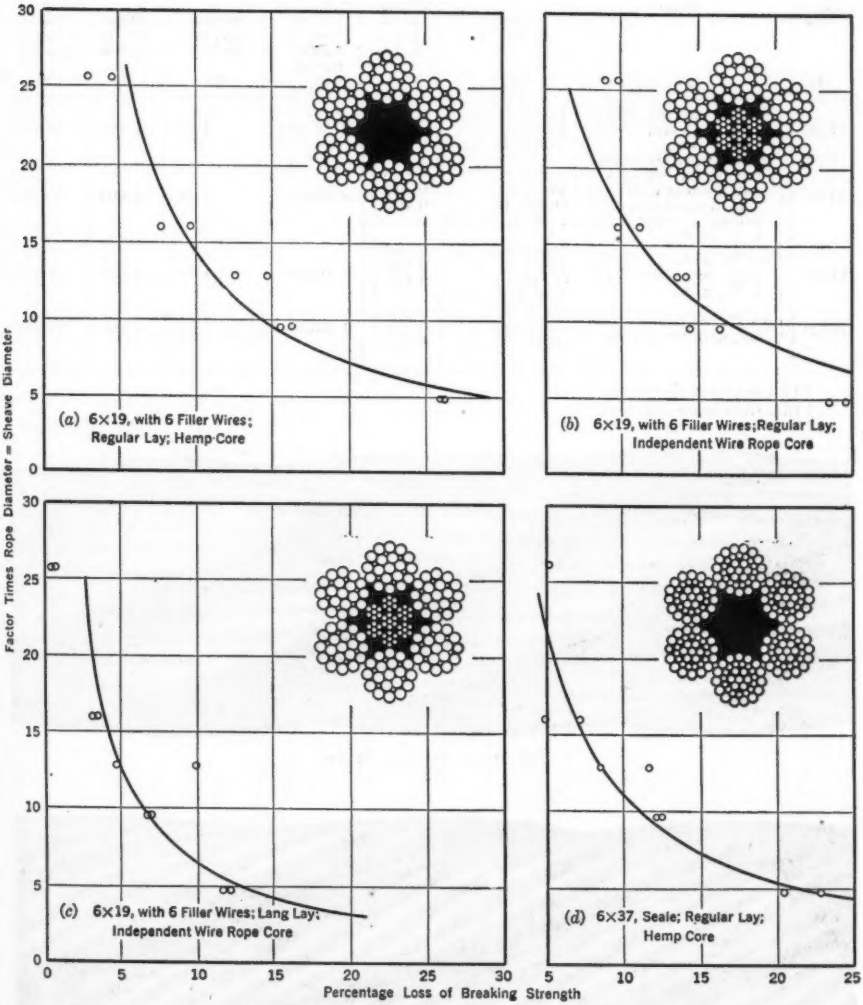

Factor Times Rope Diameter = Sheave Diameter
Percentage Loss of Breaking Strength
(a) 6×19, with 6 Filler Wires; Regular Lay; Hemp Core
(b) 6×19, with 6 Filler Wires; Regular Lay; Independent Wire Rope Core
(c) 6×19, with 6 Filler Wires; Lang Lay; Independent Wire Rope Core
(d) 6×37, Seale; Regular Lay; Hemp Core

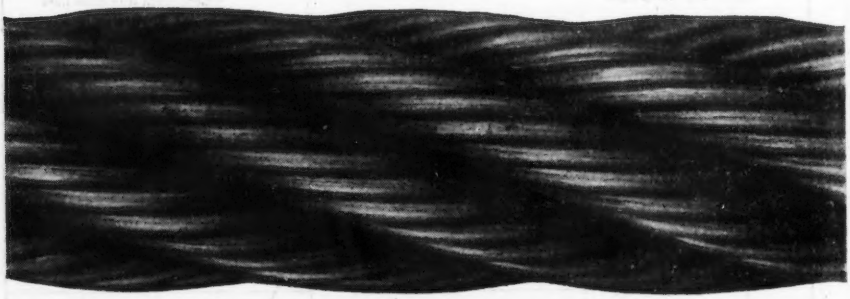
FIG. 14.—LOSS OF STRENGTH IN WIRE ROPE, DUE TO BENDING AROUND SHEAVES.

In the case of a lang lay rope the angle of lay of the wires is equal in magnitude and direction to that of the strands. The resultant angle is, therefore, $18.5 + 18.5 = 37°$, and the axes of the wires on the crowns of the strands form imaginary angles of 37° with respect to the rope axis.

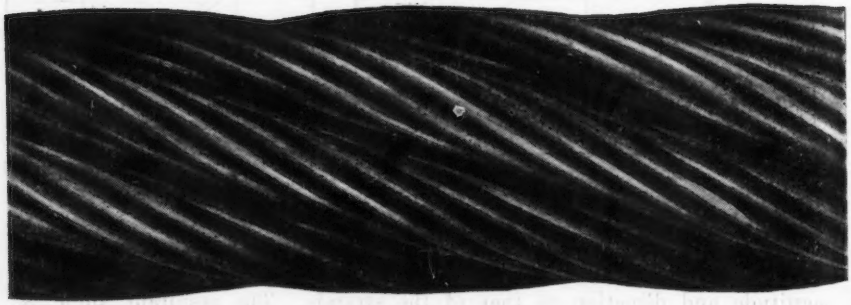
TABLE 12.—CONSTRUCTION DETAILS OF $\frac{5}{8}$-INCH IMPROVED PLOW STEEL WIRE ROPE USED FOR TEST SPECIMENS

Figure No.	Rope	Construction					Breaking load of straight specimens, in pounds
		Wires in strand	Diameter of wires, in inches	Metallic cross-section of rope, in square inches	Strand lay, in inches	Rope lay, in inches	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
14 (*a*)	6 x 19, with six fillers; regular lay, with hemp core.	Inner, 7 Filler, 6 Outer, 12	0.045 0.019 0.040	0.16341	1.56*	4.00†	33 720
14 (*b*)	6 x 19, with six fillers; regular lay, with 7 x 7 independent wire rope core (a small wire rope within a wire rope; I. W. R. C.)	Inner, 7 Filler, 6 Outer, 12 I. W. R. C., 7x7	0.045 0.019 0.040	0.19396	1.56*	4.00†	37 400
14 (*c*)	6 x 19, with six fillers; lang lay, with 7 x 7 I. W. R. C.	Inner, 7 Filler, 6 Outer, 12 I. W. R. C., 7 x 7	0.045 0.019 0.040	0.19396	1.56†	4.00†	41 175
14 (*d*)	6 x 37, Seale; reguiar lay, with hemp core.	Inner, 7 Intermediate, 15 Outer, 15	0.034 0.024 0.034	0.15779	Inner, 0.64* Main, 1.61*	4.00†	31 660

* In a left-hand direction.
† In a right-hand direction.



(*a*) REGULAR LAY ROPE.



(*b*) LANG LAY ROPE.

FIG. 15.—COMPARISON, SHOWING DIFFERENCE BETWEEN REGULAR LAY AND LANG LAY WIRE ROPE.

The breaking load of the various samples subjected to straight pulling loads were determined in each case by striking an average of several tests, and the values in Column (8), Table 12, were obtained.

The percentage loss in breaking strength of the bent specimens, cut from the same rope on which the straight load tests were obtained, have been presented as plotted points in Fig. 14.

Curved lines in Fig. 14—drawn in each case to represent the average results of the several tests—immediately suggest a relationship between the sheave diameter and the loss in breaking strength of the ropes to conform to hyperbolic curves, with gradually increasing sheave diameters, until infinity has been approached for the radius of curvature of the sheave. The breaking strength of the bent sample can readily be visualized gradually approaching the breaking strength of the straight sample until for all practical purposes the sample conforming to an infinite radius of curvature may be considered straight.

It is also possible to visualize a sheave diameter so small as to have the effect of the blade of a knife-edged cutter. A wire rope bent to conform to the radius of curvature of this extremely small sheave diameter will be severed with practically no applied tension to the two ends of the rope. The resulting values on the curves approach the two asymptotes or axes at right angles to each other in infinity.

The proper definition of a hyperbola is "the locus of a point such that the difference in distances between it and two fixed points is constant." The equation for the rectangular equilateral hyperbola, when the asymptotes conform to the abscissa and ordinate, is,

$$x\,y = \pm \frac{a^2}{2} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (58)$$

in which, a is the distance from the origin or common meeting point of the abscissa and ordinate to the point of inflection of the curve.

For the various types of wire rope in Fig. 14 the following formulas apply: In Fig. 14(a), $x\,y = \frac{(\sqrt{12^2 + 12^2})^2}{2} = 144$; in Fig. 14($b$), $x\,y = \frac{(\sqrt{13^2 + 13^2})^2}{2} = 169$; in Fig. 14($c$), $x\,y = \frac{(\sqrt{8^2 + 8^2})^2}{2} = 64$; and, in Fig. 14($d$), $x\,y = \frac{(\sqrt{10.5^2 + 10.5^2})^2}{2} = 110.25$.

Mr. Carstarphen's static formula and the foregoing empirical formulas definitely prove the large error introduced by the use of the ordinary theory of flexure of solid beams for the development of bending-stress formulas for wire rope. The development of the problem from an angle of considering a wire rope equivalent to a spring is a closer approach to accuracy. However a definite error is introduced by the use of this theory due to the discrepancy between helix characteristics of the outer and inner wires in the strands making up a wire rope.

F. C. CARSTARPHEN,[35] M. AM. SOC. C. E. (by letter).[35a]—It may be in order to remark that this paper was written as a defense of wire rope in the part it plays in industry, and, therefore, must refer to many topics in the use of wire rope that are controversial. It struck at the importance that is attached to the ratio of sheave-rope diameters as a criterion of the service life of wire rope, and called attention to the importance of velocity in such considerations.

It was hoped that the discussion would be extensive, and would cover all phases of the problems considered. Although this hope was not fully realized (more being said about subjects already familiar, than about those that are obscure), the writer is greatly pleased with the data that have been submitted, and feels indebted to those who have taken part in the discussion. The paper and its discussion is a start toward a symposium of the subject and may inspire research on some of the points raised. It is noted that the several discussers have covered and enlightened most of the obscure points in the original paper, so that but little need be said in closing the discussion.

Since the writer does not wish to assume to be true that which is to be proved, it is particularly fitting that the discussion of this paper be closed without serious comment, because the record should stand as it is written, a register of many views on an important subject. Several minor questions may be answered as follows.

Mr. Strachan assumes that there is some inherent difficulty in using Equation (20) to compute the loss of strength due to bending a rope made from different sizes of wire, around a given radius. The contrary is true, and as an illustration of the method, a computation is made of the loss of strength due to bending a $\frac{3}{4}$-in. wire rope around a sheave with a radius of 3 in. Fig. 16 illustrates the grouping of the wires in a $\frac{3}{4}$-in. Warrington,

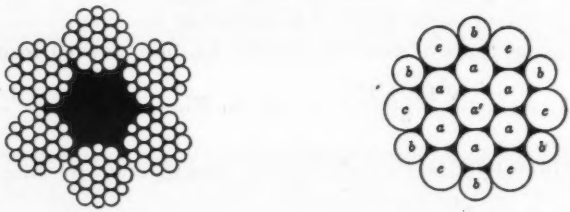


FIG. 16.—CROSS-SECTION OF 6 x 19, REGULAR LAY WIRE ROPE, WITH HEMP CENTER.

6 x 19, regular lay rope, with a hemp center. Let the seven center wires of the strand (a, in Fig 16 (b)) be round and have a diameter of 0.054 in., and the outer wires alternately $b = 0.042$ in. and $c = 0.058$ in. There are six such strands, or 114 wires. The radii from the center of the strand to the center of the several wires are as shown in Table 13.

[35] Cons. Engr., Denver, Colo.

[35a] Received by the Secretary October 28, 1932.

The diameter of the strand is 0.288 in. and that of the rope, 0.796 in. Therefore, the diameter of the core is 0.220 in.; the lay of the wires in the

TABLE 13.—CHARACTERISTIC DIMENSIONS OF A ¾-INCH WARRINGTON WIRE ROPE WITH A HEMP CENTER

Item	Description	WIRES, IN FIG. 16(*b*)		
		a	*b*	*c*
1	Number of wires per strand	7	6	6
2	Diameter of wire, in inches	0.054	0.042	0.058
3	Distance from center of strand to center of wire, in inches	0.054	0.102	0.096

strand is 1.815 in.; the lay of the rope is 4.94 in.; the angle of the wires in the strand is $\tan^{-1}\left(\frac{0.192\,\pi}{1.815}\right) = 18°23'$; and the angle of the strand in the rope is $\tan^{-1}\left(\frac{0.508\,\pi}{4.94}\right) = 17°54'$.

Applying Equation (20) to the wires forming the centers of the several strands: $r_1 = 0.254$ in.; $\alpha = 72°\ 06'$; $\sin\alpha = 0.95$ $\cos\alpha = 0.308$; and,

$$P_1 = \frac{\pi\,(0.054)^4 \times 28.5 \times 10^6 \times 11 \times 10^6}{16 \times 3.37 \times 0.254\,[2 \times 11 \times 10^6\,(1 + 0.90) + 28.5 \times 10^6 \times 0.0949]} = 13.94 \text{ lb.}$$

The pull on the six center wires (*a'*, Fig. 16(*b*)) at 13.94 lb per wire gives, say, 84 lb. Proceed to the first row of wires (*a*, in Fig. 16(*b*)) about these center wires. Since this is a regular lay rope the angle of the wires of the strand (except the center wire), with the axis of the rope is practically zero, and is so taken in this case. If it was a lang lay rope this angle would be the sum of the angles of the wires and strands, thereby demonstrating what Mr. Rairden so ably proves, that the loss of strength due to bending a regular lay rope is not the same as for a lang lay rope of the same diameter, and size of wire; thus, for Wires (*a*), Fig 16(*b*), $r_1 = 0.054$ in., and,

$$P_2 = \frac{\pi\,(0.054)^4 \times 28.5 \times 10^6 \times 11 \times 10^6}{16 \times 3.37 \times 0.054\,[4 \times 11 \times 10^6]} = 66.8 \text{ lb.}$$

The pull on thirty-six wires, at 66.8 lb per wire, gives 2 404 lb.

Then, proceed to the second row, and consider the six wires (*c*, Fig. 16(*b*)); thus,

$$P_3 = \frac{\pi \times (0.058)^4 \times 28.5 \times 10^6 \times 11 \times 10^6}{16 \times 3.37 \times 0.096\,(4 \times 11 \times 10^6)} = 48.76 \text{ lb.}$$

The pull on the thirty-six wires, *c*, at 48.76 lb per wire, gives 1 755 lb.

Finally, for the six wires, *b*, in Fig. 16(*b*):

$$P_4 = \frac{\pi \times (0.042)^4 \times 28.5 \times 10^6 \times 11 \times 10^6}{16 \times 3.27 \times 0.102\,(4 \times 11 \times 10^6)} = 12.6 \text{ lb.}$$

The pull on the thirty-six wires, *b*, at 12.6 lb per wire, gives 453 lb.

The total loss of strength due to bending the rope on a 3-in. radius (or, with a sheave ratio of $\frac{6}{0.75} = 8$), is the sum of these values or, say, 4 700 lb. If the breaking strength of the straight rope is 35 000 lb, the bending of the rope about such a sheave, statically, amounts to a loss of $\frac{4\ 700}{35\ 000}$, or 13.5% of its strength.

The following formulas for bending in wire rope should be added for completeness:

$$b = \frac{E\,d'}{D} \cos \alpha + \cos \beta \quad \ldots\ldots (59)$$

$$b = \frac{0.44\ E\ d'}{D} \quad \ldots\ldots (60)$$

$$b = \text{Equation (12)} \times \frac{2}{3} \quad \ldots\ldots (61)$$

and,

$$b = \frac{E_r\,d'}{D_1} \quad \ldots\ldots (62)$$

in which, in addition to the notation of the paper, $E_r = E'$ = modulus of elasticity of the rope, and D_1 = diameter of the bend of the center line of the rope. It is to be noted that Equation (60) is a variation of Equation (10).

Mr. Meals always writes well on any subject pertaining to wire rope, and its use, and his opinions are appreciated. The writer believes that little is gained in separating the center wire of the strand from its neighbors in estimating the strength of a wire rope.

The writer appreciates the comments of Mr. Deck and agrees with him concerning the importance of high frequency vibrations, and their effects, on stretched cables and wire ropes. This opinion is based on observations made of the performance of messenger cables in long-span transmission-line crossings of rivers and bays, such, for instance, as the great crossing of the St. Lawrence River, at Three Rivers, Que., Canada.

Messrs. Damon and Julian present a review of much that has been accepted as commonplace in discussions on wire rope and its bending stress, and, therefore, balance the discussion, and their remarks are accepted as a proper addition to it. From personal contact with the late George F. Swain, Past-President and Hon. M. Am. Soc. C. E., the writer is certain that he did not intend any sweeping generality, that may have been said concerning the constancy of the modulus of elasticity of steel, to apply to rope wire, because it is well known that the value may range between 23 000 000 and 30 000 000 lb per sq in.

The test data submitted by Mr. Rairden are most instructive in that they show conclusively, the difference in the loss of strength due to bending regular lay rope as compared with lang lay. This important fact has never

been given the attention that it merits by writers on the bending stress of wire ropes, and, of course, is ignored in the simple generalities that pass for engineering formulas of merit.

There is need for more tests of wire rope to determine the loss of strength due to bending, statically and kinetically, and no doubt these tests will be made by competent authority, but until such tests are made, there will be many opinions concerning the service factors of wire rope. Sometimes, it seems to the writer that the design of wire rope is like that of a mansion of many windows, porches, and blinds, that are shuffled around at the owner's will. There are many styles of wire rope, but all are built on the same old foundations of a hemp or wire rope center. Since such centers have little or no resiliency, it is believed that the next step forward in wire-rope construction will be in the adoption of an elastic foundation for the strands.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

APPLICATION OF DURATION CURVES TO HYDRO-ELECTRIC STUDIES

Discussion

BY MESSRS. F. KNAPP, AND ROBERT F. EWALD

F. KNAPP,[10] ESQ. (by letter).[10a]—The present tendency of engineering science leads always to the substitution of a number of isolated solutions by more general systematic methods, which do not perhaps correspond entirely to the individual properties of the case under consideration, but which by their power of general validity promise an abundance of new facts. Moreover, such a synthesis provides for a great economy of labor and time, thus fulfilling one of the engineer's obligations, namely, that of efficient and reliable service. Considering especially the mass of new hydro-electric literature, such a procedure becomes almost a necessity for the engineer who is endeavoring to keep up to date.

Any one can develop an unlimited number of variations of methods; but for daily use a general method of maximum simplicity without special exceptions combined with reasonable accuracy is required.

No doubt most young engineers bring from school an enormous quantity of scientific facts and a number of solutions of the most varied problems; but the coherence, the "Leitmotiv," is missing and the clear conception and synthesis come only after years of practical work, so that often simple fundamental questions are considered as difficult and complex.

Considering the authors' paper from the foregoing viewpoints, it seems to the writer that the assumptions made are restrictive to its general use. Large hydro-electric systems usually comprise a number of streams on widely separated water-sheds, thus contributing to the equalization of the total available natural stream flow. In any such study factors of capital importance, such as the effect of load factor and seasonal variation of load demand, as well as possibility of pondage, must be taken into account. The duration-

NOTE.—The paper by G. H. Hickox and G. O. Wessenauer, Juniors, Am. Soc. C. E., was published in May, 1932, *Proceedings*, Part 1. Discussion on this paper has appeared in *Proceedings*, as follows: October, 1932, by Messrs. Paul F. Kruse, Donald H. Mattern, James E. Stewart, Don Johnstone, L. T. Guy, and Joel D. Justin; and November, 1932, by C. Maxwell Stanley, Junior, Am. Soc. C. E.

[10] Asst. Hydr. Engr., The São Paulo Tramway, Light & Power Co., Ltd., São Paulo, Brazil.

[10a] Received by the Secretary October 6, 1932.

curve method in such cases loses much of its simplicity. The authors assume a load demand of such a nature that a uniform rate of output is required. For large storage plants such an assumption is feasible, but the actual output of run-of-river plants depends so much on seasonal system load demand, load factor, installed capacity, pondage, and seasonal distribution of flow, that this assumption may lead to considerable error. Hence, the introduction of "use curves," "utilization factors," etc.

The importance of the problem of power output of several hydro-electric developments, supplying the same power market, demands a general unrestricted solution of easy applicability. Load-duration curves and hydrograph and mass curves are the chief "members" of the general solution, avoiding, however, the "cut-and-try" process inherent in the usual weekly or daily hydrograph studies.

In 1914, Mr. C. T. Hutchinson proposed a power-energy curve,[11] which for the case under discussion has not obtained the proper consideration. According to Mr. N. E. Funk,[12]

> "The curve is constructed by plotting percentages of the total energy as abscissas against corresponding percentages of the total power or peak as ordinates. The percentage of the total energy corresponding to any percentage of the total power or peak is obtained by integrating the area of the load curve bounded by a horizontal line drawn at the desired power value through the contours of the load curve, the contours of the load curve below this horizontal line, the ordinates between the contours of the load curve and the base, and the base of the load."

This power energy curve is far more valuable than Mr. Funk's definition makes one suppose. Its value is derived from its quality as an integral curve of the load-duration curve. A thorough study of this integral curve reveals some very interesting and valuable features, among which is the possibility of reducing any load demand to a 100% load-factor demand. Mr. Hutchinson's integral curve, therefore, may be reduced to a single vertical line with sub-divisions in percentages of maximum yearly peak load. As the daily load demand varies with the seasons, a representative or equivalent daily load diagram may be chosen for each week or season. This load diagram may be reduced to 100% load factor as explained and the representative load demand lines arranged in chronological order. Thus, a two-dimensional total yearly load demand diagram is obtained, instead of a three dimensional "load demand mountain." Next adding the run-of-river plant hydrograph and reducing the ordinates to those of the corresponding 24-hour load demand, the diagram permits reading all necessary values directly or by means of mass curves. It shows, also, that deficiency and waste at the run-of-river plant may occur, so to speak, at the same time.

For example, the application of this procedure to the authors' Case 2 does not require Assumption (1) (see "Basic Assumptions"). Although a hydrograph is necessary, tedious day-by-day studies are avoided; Assumption (3)

[11] *Transactions*, Am. Inst. Elec. Engrs., Vol. XXXIII, p. 155.

[12] "Economics of Combined Hydro and Steam Power Systems," by N. E. Funk, *Transactions*, Second World Power Conference, Vol. XI, p. 281.

can also be avoided; while Assumption (2) requires a compensation pond at a convenient point between the two plants if the distance is considerable. The method proposed also avoids the authors' fundamental procedure of constructing output duration curves for exactly one complete reservoir cycle.

Thus far, the authors treated only the maximum possible output of several hydro-electric developments, supplying the same power market. The installed capacity of each station, however, must be determined by an economic study. The capacity of a run-of-river plant and, therefore, the cost of energy, is a function of the total system load demand. The greater the demand, the greater will be the capacity. However, the power company wishes not only to produce cheap power at Plant *A* or Plant *B*, but wishes to produce the total output as cheaply as possible. With expensive peak energy the most economical capacity of a run-of-river plant becomes far greater than that determined by consideration of the economic conditions of the run-of-river plant alone.

ROBERT F. EWALD,[13] M. AM. SOC. C. E. (by letter).[13a]—The uses of the duration curve have been presented in an admirable manner in this paper to determine the relative capacities of hydro-electric generating stations to be located on the same or on continuous water-sheds where natural flow varies in an identical manner.

If the storage available at economically feasible costs is large relative to the total annual run-off, the duration curve utilized in the manner proposed may be the only tool required for a reasonably accurate determination of the proper capacity. However, in most cases, in situations similar to those under consideration by the authors, the determination of the "reliable power rate" follows a study of the most economical size of storage reservoirs to be provided. That study necessitates consideration of the flow records for a period of several years and, for this purpose, the mass curve of stream flow in its various modified forms seems to be the most convenient tool. From this point, the procedure depends entirely on the set-up of the system to which the power is supplied.

In many cases, especially those in which large steam reserves are available, in order to utilize hydro-electre energy to the greatest advantage, it is best to use variable power rates in the system, depending on reservoir and flow conditions. In order to develop the proper "rule curves" it is necessary to study the entire flow record repeatedly. For this purpose, the "energy draw-down diagram" [14] is a convenient tool. Such diagrams are readily constructed in a few hours and are of special value in studying systems utilizing large reservoirs, with relatively large draw-down, long conduits with widely varying losses, and turbines with widely varying efficiencies, because all these factors, together with leakage and evaporation loss, may be fully and easily evaluated in the preparation and use of the diagrams.

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[13] Hydr. Engr., Aluminum Co. of America, Pittsburgh, Pa.

[13a] Received by the Secretary November 1, 1932.

[14] "Power Rates in Hydro Development," by Robert F. Ewald, *Electrical World*, November 8, 1924, p. 993.

In the execution of such work it appears preferable to keep the final results in the form of graphs of power available and variation in reservoir levels, because these factors enable further study "by eye" with reference to further improvement in operation. The resulting power graphs form a series of power-duration curves from which an average duration curve may be prepared. This curve, however, has a much different significance from that discussed by the authors. To enable comprehensive understanding of the possibilities of a hydro-electric system, not only should the duration curve for reliable power rates be available for the period of minimum flow, as described by the authors, but (what seems to be more important), an average duration curve based upon application of the proposed rule curves should also be available.

The writer uses a form of mass curve that is of considerable interest—the curve of cumulative deviation from mean discharge. In its variations this curve swings about a horizontal line and, therefore, is very conveniently plotted on successive letter-sized sheets which may be easily attached to each other. The curve may be used not only in studying storage possibilities, but also systematic variations in the flow. This is an important matter, which will receive more consideration in the future.

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DISCUSSIONS

GEORGE WASHINGTON BRIDGE GENERAL CONCEPTION AND DEVELOPMENT OF DESIGN

Discussion

BY C. T. SCHWARZE, M. AM. SOC. C. E.

C. T. SCHWARZE,[13] M. AM. SOC. C. E. (by letter).[13a]—In this paper Mr. Ammann has made a distinct contribution in presenting a history of the many attempts to design a safe and adequate overhead crossing of the Hudson River, at New York City. The engineering difficulties involved in such an epoch-making design—both as to length of span and capacity for traffic—were, indeed, enormous. The splendid fruition of the adopted design is its own evidence of the care with which it was prepared, even to a consideration of background involving æsthetic feeling and, also, transportation development.

Aside from the actual kinds of structures considered in this paper, it is interesting to recall designs in non-structural steel arches that have been advanced at both ends of the chronologic period of North River bridge planning. Probably one of the first of such designs (in the earlier part of the Nineteenth Century) was that of an arch composed of wooden shear blocks. Great confidence was placed upon the shearing strength of these blocks, which were to have been cut in somewhat the same fashion as one finds in Japanese or Chinese frame construction. If memory serves, no such span as 3 500 ft. was contemplated by the designer. It is a coincidence, however, that one of the latest designs for the North River bridge crossing was also an arch; this time of reinforced concrete and for a span of 3 500 ft.! It is vouched for by at least two engineers, recently returned from abroad, that a French engineer, under the impression that the George Washington Bridge design would be competitive, prepared a design of a reinforced concrete arch for this span.

In earlier designs for the bridge under consideration there was little or no thought of a location so far north. It will be recalled that when the present streets and avenues were laid out for future development, little consideration

NOTE.—The paper by O. H. Ammann, M. Am. Soc. C. E., was published in August, 1932, *Proceedings*. Discussion on this paper has appeared in *Proceeaings*, as follows: November, 1932, by E. E. Howard, M. Am. Soc. C. E.

[13] Prof. of Civ. Eng., New York Univ., New York, N. Y.

[13a] Received by the Secretary October 25, 1932.

was given to a future northward trend of traffic. It was supposed, with methods of transportation then in vogue, that traffic would be largely east or west or, from river to river. Hence, the congestion in the comparatively few north and south avenues in Manhattan. It is, therefore, highly significant that the first bridge joining New York City with municipalities in New Jersey should be located at 179th Street, roughly, a dozen miles north of Brooklyn Bridge, and not at Canal Street. In more than one instance, insistence by business people of a community to have through traffic routed so as to pass business houses, has proved to be a boomerang. Usually, considerable harm and little or no good has resulted from the ensuing traffic congestion. Thus, little or no tendency is found toward the development of a business section at the New Jersey end of the bridge, and on the New York side, a tunnel is actually in process of construction to move the major portion of bridge traffic away from the business section in 181st Street.

One phase of this paper deserves special attention. Engineers have acquired a rather unenviable reputation as being contemptuous of æsthetic feeling in their structural design. In days gone by, when it became necessary to cover structural elements, such as steel, with masonry, the covering was often not only without taste but out of harmony with the surroundings. The most beautiful engineering structures extant, Gothic cathedrals, are beautiful because the designing engineers (and they were engineers even though they were garbed in monk's gown and cowl) were so delighted with the perfected structural elements that they spent more thought and effort in emphasizing these than in any other parts of the structure. Thus, in true Gothic buildings, the structural elements are intentionally visible from both outside and inside. The first coverings of the modern steel skeleton buildings were hideous in appearance and out of harmony with the character of construction. Later methods practically begun with the Woolworth Building, emphasize the vertical as well as the horizontal structural lines and, behold, the beautiful skyscrapers! Editorial and other comment in the press and magazines during this period of development are eloquent on this point.

Designers for the Westchester County Park Commission in New York were also awake to possibilities of æsthetic appearance. Their rigid frame bridges, handsome in appearance to the engineer whose photo-elastic eye sees beautifully adjusted stresses, were made attractive and pleasing to the non-engineering eye by "fake" stone masonry covering. More than this, the engineers were very particular about the selection of local stone so that the character and color of the stone covering should be in harmony with surrounding landscapes. Mr. Ammann justly takes pride in the beauty of the design of the steel work in the towers of this bridge. Friends of the writer, critical of æsthetic effects, have expressed appreciation of the towers as they are. This pleasing appearance is particularly noticeable from the New Jersey side, when the towers are viewed with the tall Medical Center buildings as a background.

Provision has been made to cover the unethical appearance in concrete in the anchorage. It may be suggested that the stone cover be selected, as is

done in Westchester parkways, to harmonize with the rocky cliffs of Washington Heights.

Masonry covering of prominent structures should be plausible. If it is to be carried by the tower steel, the towers must be relatively narrow. Narrow masonry towers of such height (600 ft.) may prove to be unconvincing to the layman's eye (say, from a river steamer), who sees the cables resting on them and not on the reliable steel bents. A tall and narrow masonry tower, apparently carrying such a heavy and shifting load, would appear to be very weak. This may defeat, effectively, the purpose of æsthetic treatment and prove to be just another attempt at "gilding the lily."

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

TESTS OF RIVETED AND WELDED STEEL COLUMNS

Discussion

BY MESSRS. RAYMOND J. ROARK, AND LYNDON F. KIRKLEY

RAYMOND J. ROARK,[8] ESQ. (by letter).[8a]—The part of this paper that deals with the effect of welding on strains and stresses was of special interest to the writer, and it is with this subject that this discussion is concerned.

That the unequal heating caused by welding the parts of a composite member produces residual stresses is obvious, and many have expressed apprehension as to the possible effect of such stresses on the strength and stiffness of members so fabricated. Tests have been made for the purpose of securing information on this point, but few of them seem to have been planned so as to yield information on all three of the questions of practical importance, namely: (1) What total strains are produced by a given welding procedure? (2) what total stresses are produced? (3) what are the effects of these strains and stresses on the mechanical properties of the member as a whole? Thus, the tests made by Messrs. Slater and Fuller revealed the magnitude of the total strains produced and the effect—or absence of effect—on the mechanical properties of the columns; but, as the authors point out, they did not show what part of the total strain was plastic and what part represented elastic deformation accompanied by residual stress.

Some time ago the writer made tests designed to throw light on all three of the foregoing questions as they applied to the so-called "button" or rivet welding. The connection, in this type of weld, is effected by punching or drilling holes in one of the parts, placing it against another part, and filling or partly filling the holes with weld metal, as shown in Fig. 23. It is a convenient substitute for riveting, and is especially suitable for building up composite beams or colums of plates and channels.

NOTE.—The paper by the late Willis A. Slater and M. O. Fuller, Members, Am. Soc. C. E., was published in September, 1932, *Proceedings*. This discussion is published in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

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[8a] Received by the Secretary November 18, 1932.

The specimens used are shown in Fig. 24. The scheduled procedure in making up the specimens and the corresponding standard test pieces was as follows:

(1) Make six specimens from three bars; namely, Specimens *A* and *B* from one bar, Specimens *C* and *D* from one bar, and Specimens *E* and *F* from one bar.

(2) Make one standard tensile specimen from each bar.

(3) Anneal all material, including tensile specimens, after cutting.

(4) Drill Specimens *A*, *C*, and *E* for strain-gauge holes before welding and take strain-gauge readings before and after welding.

(5) Weld Specimens *A*, *B*, *C*, and *D*, thus:

(*a*) Weld Holes 1 and 5 (Fig. 24) and allow 30 min. for cooling.

(*b*) Weld Holes 3 and 6, and allow 30 min. for cooling.

(*c*) Weld Holes 2 and 4.

(6) Weld Specimens *E* and *F* without cooling and, in the sequence, Holes 1, 5, 3, 6, 2, and 4.

(7) Saw Specimens *A*, *C*, and *E*, as shown in Table 7, and then take strain-gauge readings.

(8) Make tensile tests with extensometer on Specimens *B*, *D*, and *F*, and on the standard tensile specimens.

(9) Make two additional specimens, *G* and *H*; follow the welding procedure outlined in Steps 2, 3, 4, 5, and 7, but anneal after welding and then test in tension.

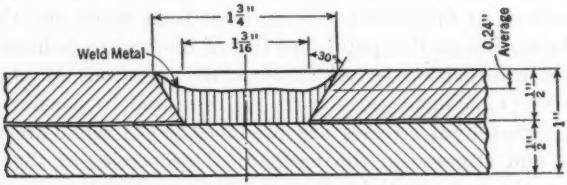


FIG. 23.—SECTION *A-A*, IN FIG. 24.

Referring to Step (7) strain-gauge readings were taken on Specimens *A*, *C*, and *E* before and after welding, measurements being made on the gauge

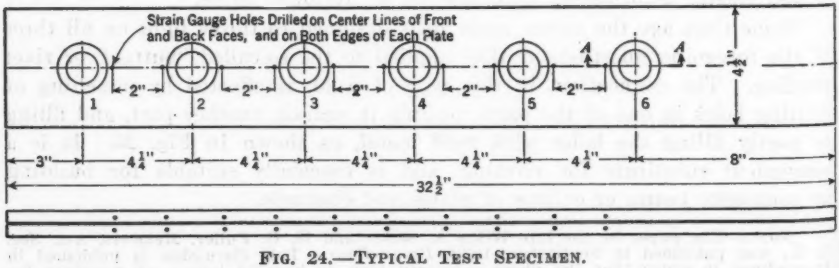


FIG. 24.—TYPICAL TEST SPECIMEN.

lines indicated in Fig. 24, and the total strain consequent upon welding was thus determined. The plates were sawed through as directed by Step 7 to

ascertain how much of this strain was elastic. The front plates (with holes) were cut on the solid lines and the back plates (without holes) were cut on the dotted lines. A third set of strain-gauge readings was then taken on the uncut plate, and the elastic recovery thus revealed made possible an approximate calculation of the actual residual stresses introduced into the plates by the welding operation. Of course, only average stresses over the 2-in. gauge lengths could be determined thus, and it was possible to ascertain neither the maximum stresses nor the manner in which the stress varied from point to point.

The results of these measurements and calculations are given in Table 7. For convenience, the product of unit deformation and modulus of elasticity is given, rather than the unit strain. When this product is calculated on the basis of total strain (plastic and elastic), it is called apparent stress and when calculated on the basis of elastic strain, it is called actual stress. The terms are not exact, but it is believed that their significance is clear. In Table 7 these apparent and actual stresses are tabulated adjacent to the gauge lines where they were found to occur.

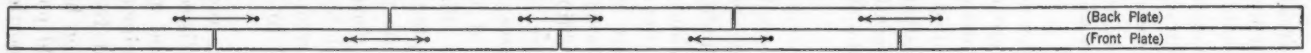
A study of Table 7 shows that: (1) The apparent stresses were very large; (2) the actual stresses were large in some cases, but were very much less than the apparent stresses, and had little if any consistent relationship thereto; (3) both apparent and actual stresses were highly irregular, showing considerable variation as between different specimens and as between different gauge lines on the same specimen; (4) in general, the actual stress in the intervals between welds was compressive in the rear plates and tensile in the front plates; and (5) as between the standard welding procedure, used in preparing Specimens *A* and *C*, and the "rapid" welding used in preparing Specimens *E*, the standard procedure appears to have caused much smaller apparent stresses and somewhat smaller actual stresses.

Five specimens were tested in tension. Three of these, *B*, *D*, and *E* were, respectively, similar to Specimens *A*, *C*, and *E*. The two others, Specimens *G* and *H*, were similar to Specimens *A* and *C*, but were annealed subsequent to welding. The quantities determined in the tensile test were the modulus of elasticity, proportional limit, apparent elastic limit, and yield point. In addition, the load at which the plaster wash began to flake off, and and the points at which such flaking first occurred, were noted. The capacity of the machine used was not great enough to permit the ultimate strength to be ascertained.

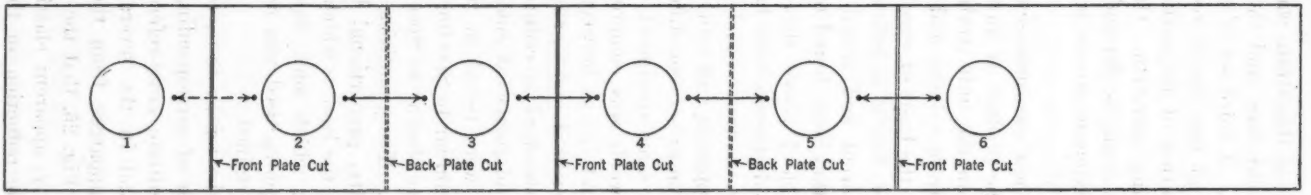
The results of the tensile tests, together with the results of tensile tests on standard coupons of corresponding material, are listed in Table 8, which is arranged to facilitate cross-reference between the actual and apparent initial stresses found in the several types of specimen and their tensile properties. It is apparent from these results, and from the stress-strain diagrams shown in Fig. 25, that the welding operation had little effect upon modulus of elasticity, apparent elastic limit, or yield point, but that it did cause a considerable reduction in the limit of proportionality, in comparison with either the annealed Specimens *G* and *H*, or the coupons.

TABLE 7.—STRESSES MEASURED IN TEST SPECIMENS, IN THOUSANDS OF POUNDS PER SQUARE INCH

Specimen	Back plate	Front plate	Back plate	Front plate	Back plate
			APPARENT STRESS; UPPER EDGE		
A	−17.2	−12.0	−15.0	−14.3	−12.7
C	−13.5	−12.0	−12.7	−11.2	−14.3
E	+14.2	+ 6.0	−39.7	−18.0	−76.5
			ACTUAL STRESS; UPPER EDGE		
A	− 2.25	+ 2.25	0	− 0.75	− 6.75
C	− 1.5	+12.0	− 4.5	+ 8.25	0
E	+10.5	− 7.75	−10.5	− 1.5	+22.5



			APPARENT STRESS; FACE		
A	+67.5	+84.0	+65.0	+76.5	+66.0
C	+49.5	+77.2	+70.0	+64.0	+60.0
E	+40.5	+82.5	+45.75	+73.0	+93.0
			ACTUAL STRESS; FACE		
A	+ 3.0	− 8.25	+ 5.25	0	+ 6.0
C	0	+ 5.25	+12.2	+10.5	+ 1.5
E	0	0	0	− 2.25	+ 6.0



			APPARENT STRESS; LOWER EDGE		
A	−30.0	−9.75	−11.2	−11.2	−12.0
C	−10.5	−5.25	− 3.75	−13.5	−12.0
E	−167.5	−9.0	+18.0	0	+42.0
			ACTUAL STRESS; LOWER EDGE		
A	−3.0	+8.25	−0.75	0	−0.75
C	−1.5	+3.0	+3.75	0	−9.75

It appears reasonable to assume that the low elastic limit of the welded test pieces was due to the presence of initial stresses, the material under initial stress reaching its elastic limit at a comparatively low average load. The rather well-defined break in each of the stress-strain diagrams would also seem to indicate that a considerable volume of material reached its elastic limit at the same time, and, therefore, must have been under an approximately uniform intensity of initial stress.

TABLE 8.—RESULTS OF TENSILE TESTS ON WELDED SPECIMENS COMPARED WITH STANDARD TENSILE SPECIMENS

WELDED SPECIMEN:		MODULUS OF ELASTICITY, IN MILLIONS OF POUNDS PER SQUARE INCH		UNIT STRESSES, IN KIPS PER SQUARE INCH						
				Proportional Elastic Limit		Useful Limit Point		Yield Point		Point at which flaking was first observed
Tested in tension	Tested for residual strain	Welded specimen	Standard tensile specimen	Welded specimen	Stanardd tensile specimen	Welded specimen	Standard tensile specimen	Welded specimen	Standard tensile specimen	
*B**	*A**	29.0	29.0	13.25	29.6	27.5	30.8	32.45	33.75	32.2§
*D**	*C**	29.25	28.3	14.25	29.6	26.2	30.8	33.0	33.75	36.7‖
F†	*E*†	27.0	29.3	13.25	27.0	28.15	31.4	33.7	34.2	36.4¶
G‡	*A* and *C*‡	....	....	27.25	....	28.25	....	30.0	....	....
H‡	*A* and *C*‡	....	....	30.0	....	30.6	....	32.5	....	....

* Welding procedure was normal; that is, time was allowed for natural cooling.
† Rapid welding.
‡ Welding procedure same as for Specimen *B*, except that specimens were annealed after welding.
§ Flaking first observed on the side opposite the welds.
‖ Flaking first observed on the side opposite Welds 4 and 5.
¶ Flaking first observed on the side opposite Welds 2, 6, and 4.

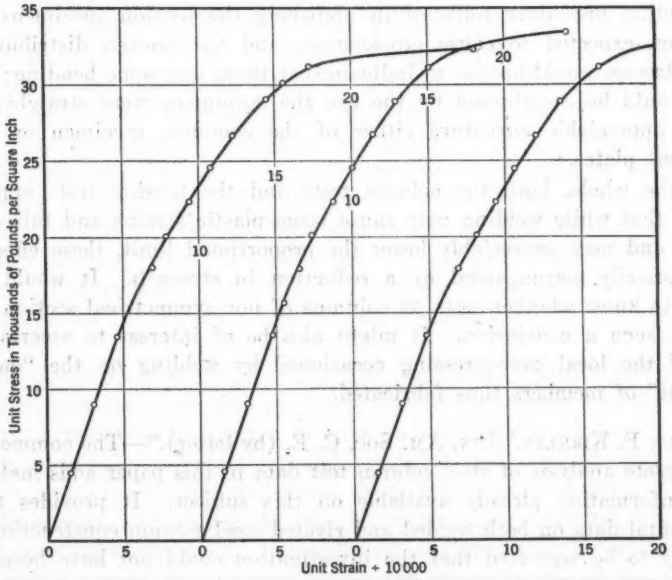


FIG. 25.—STRESS-STRAIN CURVES FOR WELDED SPECIMENS *B*, *F*, AND *D*, TESTED IN TENSION.

When the results of these tests are compared with those reported by Messrs. Slater and Fuller, it is seen that the apparent stresses are of about the same order of magnitude. The volume of weld metal deposited, and the corresponding heat input, was proportionately greater in the case of the tension specimens, and as would be expected the greatest strains caused were somewhat greater than those found in the columns. With the exception of a few values, however, the difference is not great, and it is of interest to note that the reduction in proportional limit caused by welding was about the same in the columns as in the tension specimens.

It would not be expected that the yield point in tension of a test piece as a whole would be affected appreciably by initial stresses, because the relatively large unit deformation occurring at and near the yield point permits an equalization of stress; but it might be supposed that, in the case of a column, any considerable non-uniformity in stress distribution would lead to buckling at a load less than that which the member might otherwise be expected to carry. The tests reported by Messrs. Slater and Fuller, however, fail to show any reduction in strength due to initial stress. It is to be noted, in this connection, that the columns tested by the authors were symmetrical with respect to the longitudinal axis, and that the welds also were symmetrically disposed with respect to that axis. This circumstance would tend to minimize both the tendency of the welding stresses to distort the column and their tendency to cause buckling due to local over-stressing. In a column in which neither the material nor the welding was symmetrical with respect to the longitudinal axis, both these tendencies might be apreciable. The welding procedure followed in preparing the tension specimens might have been expected to cause crookedness, and the uneven distribution of initial stresses would appear to indicate that there was some bending; but as far as could be ascertained by the eye the specimens were straight; there was no appreciable curvature either of the complete specimen or of the component plates.

On the whole, both the column tests and the tension tests appear to indicate that while welding may cause large plastic strains and fairly large stresses, and may appreciably lower the proportional limit, these effects are not necessarily accompanied by a reduction in strength. It would be of interest to know whether tests on columns of non-symmetrical section would bear out such a conclusion. It might also be of interest to ascertain the effect of the local over-stressing occasioned by welding on the "damping coefficient" of members thus fabricated.

LYNDON F. KIRKLEY,[9] JUN. AM. SOC. C. E. (by letter).[9a]—The commendable and complete analysis of steel column test data in this paper adds materially to the information already available on this subject. It provides needed experimental data on both welded and riveted steel column construction, and it is only to be regretted that the investigation could not have been more

[9] Structural Engr. Apprentice, McClintic-Marshall Corp., Bethlehem, Pa.
[9a] Received by the Secretary November 28, 1932.

extended, especially with regard to the number of test specimens of each type as well as a wider range of core sections.

Although these tests will provide valuable information for a large part of light building construction, until similar tests have been performed on the much larger specimens used in heavy building construction engineers will not know much more of the mysteries of stress action and distribution in built-up steel columns. In heavy column construction it is not uncommon to find the heaviest of rolled H-sections buried beneath 2 to 4-in. cover-plates and surrounded by an additional arrangement of rolled angles and plates.

Until improvements have been made in the accuracy of correlation of stress analysis between these more complex column sections and the simpler types, as tested and described in this paper, not much will have been added to heavy steel column data.

Items 2, 3, and 10 under "Summary of Results," emphasize another problem of steel manufacture—the effect of heat upon the parent metal. The authors have pointed out in Item 2, the effects of welding upon the parent metal local to the point of the weld, which were caused by heating and cooling. In addition, the change in metallurgical structure of the parent metal local to the weld may be of some consideration, depending on the thickness of the two welded parts, the size of welding electrode used, arc voltage and amperage, and the rate of deposition of the weld. The effects of these conditions may vary with the thickness of the welded components and the consequent dispensation of heat of welding.

Variation of the chemical composition of the weld metal itself is also possible, depending on the degree of contamination by atmospheric gases during deposition. Mr. J. C. Hodge has pointed out[16] that by varying the efficiency of the protection, either by varying the nature and thickness of surface coatings and coverings, or by changing the length of the arc, the composition of the deposited weld may be changed at will. Thus, the nitrogen content may vary from 0.015 to 0.15, depending on the degree of protection. According to Mr. Hodge, other changes in chemical composition occur during weld deposition; for example, the carbon and manganese content are considerably lowered, which indicates the extreme oxidizing conditions within the arc. Furthermore, variations in chemical composition affect the elastic properties and the behavior under plastic deformation of the various weld metals.

The physical composition of a single ingot from which structural shapes are rolled may also vary from section to section throughout a specimen. Such variations manifest themselves in such natural defects of ingots known as piping, blow-holes, segregation, and crystallization, any of which may be present and vary in size and shape from microscopic to several inches in one dimension. These defects may or may not be corrected during subsequent working of the metal. Consequently, any one or a combination of these influencing factors may account partly for the variation of the yield point

[16] *Engineering News-Record*, October 13, 1932.

among the cover-plate and core-section coupons as mentioned by the authors in Section 1 of the Appendix.

The writer is fully aware that most of the points mentioned herein are of minor consequence with respect to strength of welded and riveted steel columns as finally produced, and that they are usually taken care of by adequate mill and shop inspections; yet he feels that they are deserving of mention and are points that contribute to the explanation of variations noted by the authors.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

PRE-QUALIFICATION OF CONTRACTORS

Discussion

BY MESSRS. C. L. HALL, A. R. LOSH, AND E. G. WALKER

C. L. HALL,[9] M. AM. SOC. C. E. (by letter).[9a]—This interesting, but all too brief, discussion of the merits and demerits of pre-qualification avoids the question of the political factors involved. The construction of public works, like any other act of the State, is necessarily political in origin and in final control. It is imperative, therefore, that the political effect of any phase of public construction be considered. The writer desires to point out that political history may throw some light on the letting of contracts.

All discussion of pre-qualification is based on the theory that, in general, the owner who is to use the work to be built can best protect his own interests by restricting competition to contractors whom he knows are capable of doing the work. In engineering matters, the wise owner follows the advice of an engineer whom he has selected. Usually, the elimination, through error, of a qualified contractor will cost the owner only the comparatively small sum of money representing the difference between the successful bid and the bid which the eliminated contractor would have submitted. On the other hand, even with excellent inspection, an unqualified contractor can sometimes cause an owner very large losses. In almost every case the relatively slight sums he can save the owner by his lower bid at least are balanced by the diminished value of any structure upon which the construction work has been below par, or by the increased cost thereof, if the inspection cost has been unduly high. If the odds are thus weighed, and the owner is assumed to be capable of exercising an intelligent care of his own money, an iron-clad case can be made out for pre-qualification.

In works of any magnitude, however, the owner is usually an abstraction, such as a corporation. In public works, the owner is so much of an abstraction, that he cannot really be identified at all. In order to visualize him those

NOTE.—The paper by C. J. Tilden, M. Am. Soc. C. C., was presented at the meeting of the Highway Division, New York, N. Y., January 22, 1931, and published in September, 1932, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: November, 1932, by Messrs. Edward W. Bush, A. B. Edwards, Samuel T. Carpenter, and Leon F. Peck.

[9] Lt.-Col., Corps of Engrs., U. S. A., U. S. Engr. Office, Cincinnati, Ohio.

[9a] Received by the Secretary November 7, 1932.

who have dealings with any particular branch of the owner government as to any work, usually refer to the bureau or State commission in charge as the owner. These organs do not resemble very closely the wise private owner.

During the scandals of re-construction, a half century ago, the contracting officer did not usually yield to crude bribery, but he frequently exercised his official discretion for the benefit of the political organization which, rather than the abstract public, was responsible for his appointment. He thus tended to award contracts for the benefit of contractors, who presumably rewarded the political organization with a share of the profits. It was the scandals that followed this relationship which caused the passage of the strict laws relative to open bidding on public works, now generally in force both in the Union and in the States.

Federal specifications, and most State specifications, even in States that have been most conservative in modifying the general laws, give a contracting officer the right to reject the bid of a contractor who lacks the requisite plant or experience. Rejection after bids are opened, or post-qualification, however, involves many inconveniences to the contracting officer. He may be accused of favoritism, or even of corruption. If the protesting bidder has, or can arouse, political or press influence in his behalf, the officer may be reversed by his political superiors. It is simpler to prevent such difficulties by pre-qualification.

With the present improved standards of honesty among public officials, there is no doubt that pre-qualification does little harm at this time; but it is only human nature for contractors to form a trade association in their own behalf. They have, assuredly, many data to support claims that inexperienced contractors injure the public. Once such an association is formed, it will also be human nature for it to grant new admissions reluctantly. If the association allies itself with employee bodies containing a large number of voters, and if the contractors play politics, the closed contractor list may become a reality.

Professor Tilden's conclusions point out some of the dangers of the closed list, even on the assumption that contracting officers act with complete independence, and in perfect good faith; but complete independence is rarely possessed by non-political officers; nor is good faith always a quality of political appointees. Moreover, the permanent officer, who is most likely to possess both these desirable characters, is by nature a conservative. He prefers to deal with contractors whom he knows. Everything thus conspires to keep a closed list closed.

Humanly speaking, it is certain that a closed contractor list will cause a repetition of the very evils that strict public works laws were designed to suppress. It is at least probable that pre-qualification will produce such a list. Post-qualification in the light of day, as is the present practice of the Federal Government, does not cause these evils. This system is possible if the State has a permanent engineering force, with an *esprit de corps* of its own, and if local public opinion refuses its support when the engineer is over-ruled by the politician on technical matters. These may be counsels of perfection;

on the other hand, without such a public organization, it will be impossible to prevent scandals following pre-qualification. The writer, therefore, concludes that statutory pre-qualification would cause disadvantages outweighing its undeniable advantages.

A. R. Losh,[10] Assoc. M. Am. Soc. C. E. (by letter).[10a]—This is a timely subject and one worthy of careful consideration. The large volume of public works contracts now under way (1932) and the dependence of the construction industry upon such contracts, make it necessary that the best business principles shall prevail. An efficient, qualified, responsible contractor is just as essential as an engineering organization with the same qualifications. Professor Tilden has presented the general features of pre-qualification in an excellent paper, and the writer would like to add a few brief comments, particularly with reference to the advantages of pre-qualification.

There appear to be a number of advantages not brought out in sufficient detail by the author. The administrative authorities on public works are relieved of the responsibility and inconvenience of assisting in collecting unpaid bills from contractors. They are relieved from the criticisms which always follow in the trail of a bankrupt contractor, of merchants, material men, laborers, and others who may sell their goods or services to the contractor. Many people who are not informed as to the provisions of public contracts and surety bonds, will sell contractors or their employees almost anything, feeling that the State or other public authority will protect them. They also feel that an organization that is qualified to receive a contract on public work must have a first-class credit rating.

Surety bonds usually protect the State in securing work within the specifications and time limits, but they do not fully protect the public from losses due to the failure of the contractor to pay his bills. Laborers and small businesses cannot afford to employ attorneys and go into Court to collect their accounts. There are occasions without number where contractors, and subcontractors, or employees, buy goods or obtain services that are not directly related to public contracts, but the uninformed public is not aware of these conditions and is without protection.

Surety companies occasionally fail, and the public is left without protection. The public official cannot specify the surety, but must accept any that is offered as long as it has a satisfactory rating. All surety companies do not handle contractors' claims on the same basis.

Companies that sell materials and equipment will be assured of receiving payments for their goods and will compete for business, resulting in a lower cost to the public. Contractors in good financial standing, who are in position to discount their bills, frequently buy cheaper than the State, while those contractors whose credit is in doubt, must pay a higher price.

Pre-qualification will have a stabilizing affect on the construction industry which, in turn, will permit the development of better organizations, more efficient work, and, in the end, reasonable prices. Certainly, on public work,

[10] Mgr., Asphalt and Road Oil Div., Anderson-Prichard Corp., Oklahoma City, Okla.

[10a] Received by the Secretary November 10, 1932.

prices should not be unreasonably high, or low. Fair prices for materials, living wages, a reasonable profit for the contractor, and the work done as required by the plans and specifications, are the desirable ends to be obtained in the termination of a public works contract.

On the side of disadvantages of pre-qualification, the need is only to guard against restricted competition. To offset this there is always left open to the administrative authority the threat to do the work with its own organization if the bids exceed the engineer's estimate.

E. G. WALKER,[11] M. AM. SOC. C. E. (by letter).[11a]—The wisdom of adopting some form of pre-qualification is apparent. Stripped of its special signification, the author's statement to this effect merely expresses a fundamental principle of commercial dealing which recognizes that price is only one of the guiding considerations in entering into a business transaction. Unfortunately, the plain common sense of a private deal has a tendency, when applied to the control of public undertakings, to become involved in an elaboration of laws, rules, and red tape. While these are devised primarily for the avoidance of the possible temptation of unethical practice, they also have the tendency to replace the responsible exercises of judgment by the mechanical application of rules. There cannot be any doubt of the practical necessity of a preliminary assessment, by the engineer responsible for the project, of the suitability of the proposed contractor, but there is grave doubt as to the propriety of working strictly by rule in making such assessment. There are many qualifications and attributes of a contractor which may make or mar the execution of a contract, and which are not susceptible of expression in a questionnaire. It is not desirable, therefore, to deal with this matter by formal rules of procedure.

It is the essence of public control that there must be public knowledge of the intention to let contracts for public works and services, and that the terms of these contracts must be fulfilled in the manner that is most economical to the community. The public interest, however, is not served by awarding contracts to the lowest bidder irrespective of other considerations. Hence, there must be some form of limitation in order that the responsible authority may fulfill its function of obtaining the best results for the public it represents. It should be advised, therefore, by a competent engineer who should be free to investigate the claims of competing contractors, and to make recommendations supported by reasoned arguments. In considering his report, the engineer should be free to make such inquiries as appear germane to the issue, and to form his opinions without the limitations of a schedule. In spite of the development of the large contracting company with its complex and sub-divided organization, the successful execution of a public works contract is, to-day, as much dependent upon the individualities of the men in control, as it was in the days of the private individual who made a personal contract for executing works. This must be taken into account adequately in comparing the proposals of competing contractors.

[11] (Maxted & Knott), London, England.

[11a] Received by the Secretary November 30, 1932.

The prime duty of the engineer is to endeavor so to serve his client as to obtain the best possible results for him within the limitations of the proposition. In order to do this effectively, the engineer has frequently to settle details which involve the striking of a fair balance between the claims of the client and those of the contractor. To do so successfully an evenness of judgment must be developed, which, if properly applied in the initial stages of the making of the contract, will go a long way to facilitate the execution of the work for all concerned therewith. After some years of experience of both public and private works, the writer submits the following three suggestions:

1.—Where the nature of the work and of the interests for whom the work is to be executed, make public advertisement of the proposed contract essential, it should be open to all contractors to tender; but essential safeguards should be inserted to make clear that the work will not necessarily be awarded to the lowest or to any bidder and, by the requirement of suitable deposits and securities by contractors, precaution should be taken to insure that only proper and serious bids are made.

2.—In cases where public announcement of the proposal is not necessary, it should be within the power of the engineer to invite an agreed number of selected contractors to bid, the engineer having first satisfied the authority responsible for the work as to his reasons for selecting the particular contractors.

3.—In any case, after bids have been received, it should be the duty of the engineer to make such inquiries in any direction as he should think fit, in order to assure himself of the suitability of the contractor before recommending the acceptance of his bid. Naturally, such an investigation would have to be comprehensive or simple, according to the knowledge available from previous transactions. It is essentially a matter in which the procedure in each individual case must be evolved at the time, and cannot be met by the filling in of a pre-arranged form.

It does not seem that the adjudication on the suitability of a contractor to undertake work for which he may compete, can be controlled in detail satisfactorily by legislation, with all the rigidity and limitations which must inevitably accompany it. The subject should be treated in a far more elastic manner. The most suitable treatment is obtained by the employment of competent and honest engineers, and by leaving them full responsibility as outlined herein, subject to the control of the responsible authority. The code of ethics of the Society is a sufficient safeguard to any community served by competent qualified engineers.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

TESTS FOR HYDRAULIC-FILL DAMS

Discussion

BY CHARLES H. PAUL, M. AM. SOC. C. E.

CHARLES H. PAUL,[17] M. AM. SOC. C. E. (by letter).[17a]—The author is to be commended both for the purpose of his paper, and for the careful detail in which the proposed tests are described. Engineers interested in hydraulic-fill dam construction will find this paper valuable for study and for reference.

As Consulting Engineer during the last fifteen months of hydraulic-fill construction, the writer made regular visits to the Cobble Mountain Dam and had the opportunity to observe the ingenuity and competence of the engineers in developing practical methods of making tests for control and records of hydraulic-fill operations.

While the hydraulic-fill method of dam construction has been in use for many years, it is true that systematic study of the details of design and construction peculiar to this type has been made only within the last decade or two. Even to the present time there are few accepted standards available for investigating or testing materials, or for determining results as the work proceeds.

It is an accepted fact that there is a wide range of materials suitable for hydraulic-fill construction, as to shoulder material as well as core material. This may be seen by a study of the descriptions of the many successful dams of that type that are now (1932) in service. Progress in standardizing testing methods unquestionably will lead to more careful consideration of the requirements of design to fit the materials available. Therefore, it is quite probable that this paper may lead to an indirect result fully as important as its main purpose.

Borrow-pit sampling begins with methods generally accepted for the investigation of materials for any earth dam, but must be carried much further. Core material available is limited approximately to the quantity passing the

NOTE.—The paper by Harry H. Hatch, M. Am. Soc. C. E., was presented at the Joint Meeting of the Irrigation and Power Divisions, Yellowstone National Park, July 7, 1932, and published in October, 1932, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[17] Cons. Engr., Dayton, Ohio.

[17a] Received by the Secretary November 12, 1932.

200-mesh sieve. An excess over apparent requirements is necessary, as some of the fines will be trapped in the beaches, and it helps core control to be able to waste some fines.[18] After the amount available has been determined and found to be sufficient, various core tests should be made for specific gravity, gradation, percentage of voids, permeability, etc., methods for which are outlined in the paper. The gradation of the coarse material in the borrow-pit should also be investigated (this may be done by screens and sieves), as the stability and free drainage of the shoulder material is fully as important as the impermeability and satisfactory rate of consolidation of the core.

The author describes the usual method of determining moisture content and percentage of voids in core and beach materials. This is a simple procedure and gives satisfactory results. His Fig. 2 shows the percentage of voids decreasing with the distance from the center line of the core. At first, the relatively high percentage of voids in the core may appear to be illogical, but that seems to be characteristic of such materials. It should be remembered that the voids in the core are microscopic, and even if they comprise a large percentage of the mass, their extremely small size renders the mass practically impervious. By reference to another section of this paper (see under "Effective Size"), it may be inferred that the rate of permeability decreases as the square root of the size of voids. Several samples of cores from the Miami Conservancy dams averaged 40% voids (weight, 125 lb. per cu. ft.; specific gravity of solids, 2.67), but the coefficient of permeability through that material was found by Glennon Gilboy, Jun. Am. Soc. C. E., to be only 0.0144 cm. per day. These void determinations on the Miami Conservancy cores were obtained from samples taken from the cores after seven years' consolidation, and, therefore, tend to confirm the results obtained by the author at Cobble Mountain. These results are further confirmed by records at many other hydraulic-fill dams.

In making and recording void determinations, care should be taken to make clear whether the amount of moisture is determined and expressed as a percentage of the entire weight (water plus solids), or of the solids alone. For example, in a sample of saturated core weighing 125 lb. per cu. ft., a water content of 25% of the mass by weight would mean 50% voids, while 25% of the dry or solid matter would mean 40% voids. There has been some confusion in the past because of lack of care in expressing or interpreting the results of moisture content determinations. The writer prefers the practice of expressing moisture content in percentage of the entire weight. That is the meaning of w in the author's Equation (1).

The writer has expressed the opinion repeatedly that rod penetration tests as ordinarily made from a boat give only indefinite results.[19] They are of some value in determining in a general way the rate of consolidation, and in detecting the presence of tongues of sand or gravel that may have worked out into the core. The author has described a more refined method of making these tests, which seems to give fairly reliable results.

[18] *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), pp. 1181, 1183, and 1236.
[19] *Loc. cit.*, Vol. LXXXIII (1919–20) p. 1800, and Vol. LXXXV (1922), p. 1193.

Several methods of determining grain sizes and gradation of core materials have been described by Mr. Hatch, including the hydrometer method. The writer agrees with the conclusion that this method has many advantages, such as speed, low cost, convenience, and reasonable accuracy. The procedure is outlined in detail, step by step. It is hoped that this hydrometer method may be adopted for general use in the investigation of borrow-pit materials and in hydraulic core tests.

With reference to suitable sizes of core materials, it may be said in general that while the presence of impalpably fine particles makes for less permeability, it tends to increase the difficulties of construction. Character of borrow-pit materials available, study of core gradations and construction records at other hydraulic fills, and good judgment must contribute to the decision in each case as to whether the hydraulic-fill type is practical and economical.

In a paper describing core studies at the Miami Conservancy Dams,[30] the writer gave analyses of representative samples of the cores of those five dams, all of which averaged much finer in sizes of particles than the Cobble Mountain core. The Miami core material averaged about 0.002 mm. for effective size and about 15 for uniformity coefficient, using those terms for want of something better to indicate fineness and gradation for purposes of comparison. Observation and tests show those cores to be highly impervious, and their rate of consolidation was fast enough for all practical purposes. Samples of the core taken after consolidation averaged 40% voids and weighed 125 lb. per cu. ft. The average specific gravity of the particles was 2.67, and that of the mass, 2.00. The material of the Cobble Mountain core had an effective size of about 0.009 mm. and a uniformity coefficient of about 8. Although much coarser than that of the Miami Conservancy, this material also gave satisfactory results as regards water-tightness, stability, and rate of consolidation. At some dams where cores were somewhat finer than those of the Miami Conservancy District, more or less trouble has occurred during construction, and not many cores are much coarser than at Cobble Mountain; but between the wide limits indicated by those two, it is reasonable to assume that core materials will be satisfactory unless preliminary tests point to the contrary. The author has wisely called attention to the fact that frequent gradation analyses are necessary to obtain a representative gradation curve at any dam. The hydrometer method, as described, makes this comparatively easy and practicable. His method of plotting these curves is believed to be logical, and may well be standardized.

The author's conclusions regarding the coefficient of friction of beach and shoulder materials (see text following Equation (18)) seem logical, and may be put into practical application in the design of the shoulder sections of hydraulic-fill dams, although in many cases other considerations, such as angle of repose, may require a volume of shoulder material that will eliminate the necessity of more than a passing consideration of the coefficient of friction.

[30] "Core Studies in the Hydraulic-Fill Dams of the Miami Conservancy District," by Charles H. Paul, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922) pp. 1181–1236.

Percolation tests are difficult to make and to interpret for practical application. Nevertheless, such tests are extremely desirable in the investigation of earth dam materials and the writer knows of no more practical apparatus for such determinations than the one described as the "Large Percolator." While it may be difficult to make a direct application of the results of such tests, they certainly furnish valuable information for comparisons of different materials available, or for possible variations in gradations. These tests, either on the job or at some available laboratory, are considered to be well worth while. As would be expected, a conclusion from the author's investigation is that both finer grain sizes (which means smaller void sizes), and lower percentages of voids, have a strong influence in reducing permeability.

As to rate and manner of core consolidation, the experience with the Miami Conservancy District cores heretofore described, showed that they reached a condition very close to final consolidation in not more than twelve months after placing. The rate of construction progress has a marked effect on this, of course, since the faster the load is applied the more rapid is the consolidation process. In this connection (referring to the discussion involving Equations (25) to (30)), the writer is convinced that considerable of the core drainage takes place vertically.[21] The writer belives that this vertical drainage will have even more effect than is suggested by the author.

Notwithstanding the difficulties in applying results of percolation tests o actual conditions in a dam, nevertheless such tests may often suggest limits that will be safe to apply to the dam itself. In such cases the author's seepage formula (Equation (43)) is set up in comparatively simple and convenient form for use. It should be realized also that this formula is applicable to the seepage-resisting materials in any type of earth dam, and not to hydraulic cores only.

It is the writer's belief that a well-built hydraulic core, composed of materials within the limits of fineness being discussed in this paper, will scarcely permit enough seepage to cause any concern. From the nature of their construction, hydraulic-fill dams are particularly adapted to resist seepage through the core itself, and to distribute such seepage harmlessly and effectively through the beach and shoulder materials on the down-stream side. Assuming that the core is homogeneous, what little seepage may occur will be taken care of automatically. One thing to be watched carefully is the preservation of uniformity in the core during construction, and the prevention of tongues or layers of sand or gravel that will provide a path for free seepage. Seepage formulas must necessarily assume the absence of any such conditions as that.

In many cases, also, it will be found that the dam itself is more watertight than the abutments against which it rests. Seepage dangers should be investigated through abutments and foundations of earth dams, as well as through the dams themselves, as otherwise these abutments or foundations may be the source of the greater danger.

[21] *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), pp. 1194, 1199 (Fig. 11), and 1201.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

A HISTORY OF THE DEVELOPMENT OF WOODEN BRIDGES

Discussion

BY HENRY B. SEAMAN, M. AM. SOC. C. E.

HENRY B. SEAMAN,[41] M. AM. SOC. C. E. (by letter).[41a]—This valuable paper is not merely a history of wooden bridges; it is practically a treatise on the design of wooden bridges of the more recent types. The Howe truss will be in vogue in timber countries for many years to come. The recent tendency toward iron bridges is partly the result of heavier engine loading; but it is due more especially to the increased price of timber, which would have less effect in countries where the timber is ready at hand.

Fifty years ago there were still many Howe truss bridges on the Pennsylvania Railroad. Although they were really combination bridges of wood and iron, they were classed as wooden bridges, and if iron was used unnecessarily it was considered an indignity to the wooden truss. Cast-iron angle blocks and wrought-iron tension rods were a regular part of the construction. The chord splices were of oak and were compactly bolted in place. In proportioning the splice, the area of the split section to that in the end thrust bearing was in a fixed ratio. Long spans, which still remained in service, had arch ribs on each side of the truss, which extended down below the chord to the face of the abutment, on skew-backs.

On the Erie Railroad, about ten years later, there were many Howe truss bridges of shorter span, but without the arch rib. Since the Railroad Company was avoiding the appointment of a Receiver, the bridges were used to their ultimate of safety and were badly "hog-backed" before removal. They were tested at the critical joints by the use of a small auger to ascertain the condition of the interior wood, and the holes thus made were then filled tight with plugs. It was the saying that "a wooden bridge never falls down," which was, of course, a dangerous doctrine; but it received apparent corrobo-

NOTE.—The paper by Robert Fletcher and J. P. Snow, Members, Am. Soc. C. E. was published in the November, 1932, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[41] Cons. Engr., Brooklyn, N. Y.

[41a] Received by the Secretary November 28, 1932.

FIG. 29.—TIMBER HOWE TRUSS, MOUNT VERNON, N. Y., 1893.

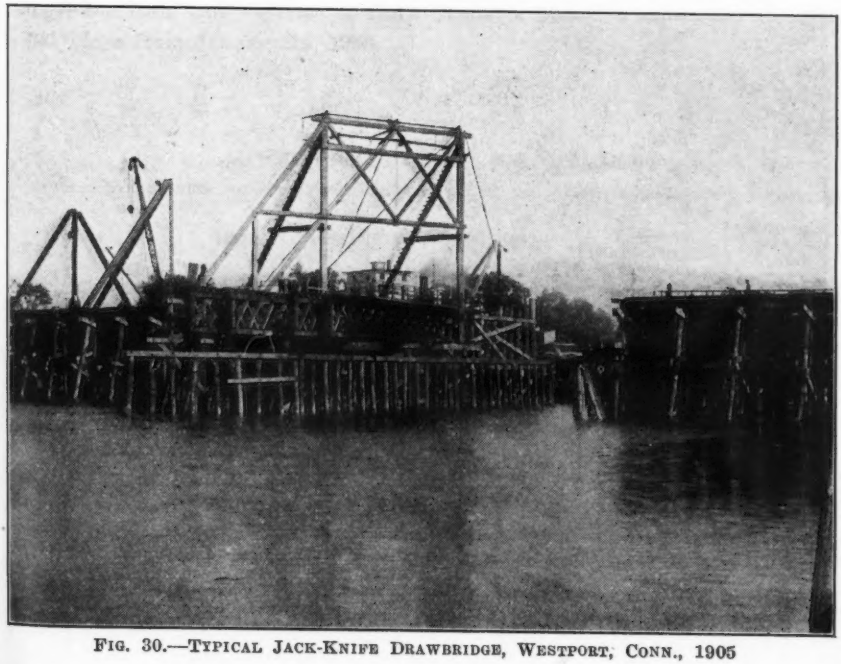

FIG. 30.—TYPICAL JACK-KNIFE DRAWBRIDGE, WESTPORT, CONN., 1905

ration when the Market Street Bridge, in Philadelphia, Pa., was removed, and was found to have been literally punk.

About 1894, the writer had occasion to design and construct the overhead Howe truss bridge shown in Fig. 29, for a temporary crossing of the New York, New Haven and Hartford Railroad, at Mount Vernon, N. Y., and found no difficulty in obtaining trained carpenters to execute the work. It was of simple design, with oak angle blocks, a complete upper lateral system, and knee-braces only at the portals.

More could be written on the subject of the jack-knife draw-bridge. This is still a useful type for temporary purposes. While re-constructing the main-line bridges of the New Haven Railroad at Cos Cob and at Westport, Conn., in 1905, two such bridges, double-tracked, were constructed by the writer, and served well for main-line service during the reconstruction. One of them is shown in Fig. 30. These jack-knife draw-spans were part of the temporary by-pass trestles. For satisfactory operation of the draw-span, it is important that the gallows-frame be inflexible, and that the pile bents from which the trusses swing, be rigidly braced against movement.

APPLICATIONS FOR ADMISSION AND FOR TRANSFER

The Constitution provides that the Board of Direction shall elect or reject all applicants for *Admission* or for *Transfer*, and, in order to determine justly the eligibility of each candidate, the Board must depend largely upon the Membership for information.

This list is issued to members in every grade for the purpose of securing all such available information, and every member is urged to scan carefully each monthly list of candidates and to furnish the Board with data in regard to any applicant which may aid in determining his eligibility. It is the *Duty* of all *Members* to the *Profession* to assist the *Board* in this manner.

It is especially urged, in communications concerning applicants, that a *Definite Recommendation as to the Proper Grading in Each Case* be given, inasmuch as the grading must be based upon the opinions of those who know the applicant personally, as well as upon the nature and extent of his professional experience. If facts exist derogatory to the personal character or to the professional reputation of an applicant, they should be promptly communicated to the Board. *Communications Relating to Applicants are considered by the Board as Strictly Confidential.*

The Board of Direction will not consider the applications herein contained from residents of North America until the expiration of thirty (30) days, and from non-residents of North America until the expiration of ninety (90) days from January 15, 1933.

MINIMUM REQUIREMENTS FOR ADMISSION

Grade	General Requirement	Age	Length of Active Practice	Responsible charge of work
Member	Qualified to design as well as to direct important work	35 years	12 years*	5 years
Associate Member	Qualified to direct work	27 years	8 years*	1 year
Junior	Qualified for sub-professional work	20 years†	4 years*	
Affiliate	Qualified by scientific acquirements or practical experience to co-operate with engineers	35 years	12 years*	5 years
Fellow	Contributor to the permanent funds of the Society			

* Graduation from a school of engineering of recognized reputation is equivalent to 4 years active practice.

† Membership ceases at age of 33 unless transferred to higher grade.

The fact that applicants give the names of certain members as references does not necessarily mean that such members endorse.

FOR ADMISSION

BALDWIN, EDWARD WILLIS, JR., DeKalb, Ill. (Age 22.) Refers to H. E. Babbitt, J. J. Doland, W. C. Huntington, G. W. Pickels, T. C. Shedd.

BOBISCH, WILLIAM JULIUS, Chicago, Ill. (Age 22.) Refers to J. J. Doland, W. C. Huntington, G. W. Pickels.

BOONE, HARRY KAY, Los Angeles, Cal. (Age 35.) Archt. Engr. Refers to W. H. Baker, Jr., P. H. Calahan, J. D. Faulkner, W. J. Fox, C. C. Miller, F. E. Trask.

CHASE, LOVELL CONVERSE, Waterville, Me. (Age 22.) Instructor in Mathematics, Coburn Classical Inst. Refers to H. S. Boardman, E. H. Sprague.

COLLINS, CHESTER PAINE, Los Angeles, Cal. (Age 42.) Res. Engr., Los Angeles County Road Dept. Refers to R. L. Anderson, E. A. Burt, O. F. Cooley, G. W. Jones; F. H. Joyner.

CRUMLISH, HUGH JOSEPH, Pleasant Gap, Pa. (Age 37.) Constr. Engr., Western State Penitentiary, Bellefonte, Pa. Refers to J. T. Campbell, D. E. Davis, F. C. Foote, J. R. Hoffert, F. Holmes, M. G. Mansfield, A. J. Warlow.

DIKE, OSCAR DERMIT, Seattle, Wash. (Age 27.) Refers to I. L. Collier, C. W. Harris, G. E. Hawthorn, C. C. More, R. G. Tyler.

ENNIS, CHARLES WILBUR, JR., Houston, Tex. (Age 22.) Refers to J. J. Doland, H. H. Jordan.

EVANS, SAMUEL ANTHONY, Los Angeles, Cal. (Age 38.) Jun. Civ. Engr., Dept. of Water and Power. Refers to C. E. Angilly, Jr., W. W. Hurlbut, H. L. Jacques, F. D. Pyle, S. L. Rothery, H. A. Van Norman, W. W. Wyckoff.

GEYER, JOHN CHARLES, Cambridge, Mass. (Age 26.) Student and Asst. Instructor in Hydraulics and San. Eng., Harvard Eng. School. Refers to R. A. Dodge, G. M. Fair., A. Haertlein, L. J. Johnson.

GLASSMIRE, HARRY MARTIN, Buffalo, N. Y. (Age 57.) Asst. City Engr. Refers to J. H. Feigel, G. F. Fisk, C. L. Howell, G. S. Minniss, J. T. Mockler, W. A. Rathmann, G. F. Unger.

GOLDMAN, PERRY JOSEPH, Philadelphia,, Pa. (Age 24.) Asst. Chf. Engr., Golder Constr. Co., Engrs. and Contrs. Refers to H. N. Benkert, L. Costello, M. Golder, H. H. Quimby, C. H. Stevens.

GROVER, NEWELL ARTHUR, San Francisco, Cal. (Age 33.) Draftsman, California Dept. of Highways. Refers to J. E. Allen, S. V. Cortelyou, N. C. Grover, G. W. Rear, G. A. Wood.

HEDGES, WARREN BARTLETT, Springfield, Mo. (Age 33.) Supt. with Hedges-Weeks Constr. Co. Refers to B. L. Brown, J. S. Hazelton, F. G. Jonah, H. P. Mobberly, L. F. Van Hagan, L. A. Weeks.

HENSLEY, JAMES CARLISLE, Abilene, Tex. (Age 33.) County Engr., Taylor County. Refers to G. Gilchrist, C. S. Henning, Jr., G. R. Johnston, T. D. Shiels, W. J. Van London.

HO, CHITTY, Iowa City, Iowa. (Age 30.) Research work, State Univ. of Iowa. Refers to F. M. Dawson, B. J. Lambert, F. T. Mavis, F. A. Nagler, E. W. Schoder, S. M. Woodward.

HUGHES, HENRY CLAY, Corpus Christi, Tex. (Age 34.) Div. Res. Engr., Maintenance Dept., Texas State Highway Dept. Refers to T. W. Bailey, C. M. Blucher, E. N. Gustafson, T. E. Huffman, C. W. McFerron, R. W. Stiles.

JOHNSON, ELLERY FARMER, St. Louis, Mo. (Age 26.) Engr., International Shoe Co. Refers to E. O. Sweetser, J. L. Van Ornum.

KUHNS, JOHN DANIEL, Springfield, Ohio. (Age 33.) Business Mgr., Board of Education. Refers to T. D. B. Groner, J. P. Reinheimer, C. H. Shea, C. H. Shook C. W. Ullom.

MALTMAN, ALEXANDER, Philadelphia, Pa. (Age 46.) Acting Asst. Engr., Dept. of City Transit. Refers to J. E. Boatrite, S. Harris, V. L. Havens, H. S. Hipwell, S. W. Lamborn, C. W. Palmer, C. H. Stevens.

O'HARRA, WAYNE GILDER, Phoenix, Ariz. (Age 29.) Chemist and Office Asst., Arizona State Highway Dept. Refers to H. H. Brown, R. A. Hoffman, W. R. Hutchins, E. V. Miller, J. S. Mills, J. W. Powers.

RANKIN, TOM HARRISON, Flushing, N. Y. (Age 28.) Asst. to Chf. Engr. for Triest Contr. Corporation, New York City. Refers to F. L. Cranford, J. I. L. Hogan, H. Holbrook, H. O. Locher, J. C. Meem, H. F. Peckworth, W. G. Triest.

RICCIO, JOSEPH GEORGE ANTHONY, Bristol, R. I. (Age 23.) Refers to C. D. Billmyer, J. L. Murray.

ROLIN, RAYMOND GUSTAF, Cambridge, Mass. (Age 24.) Asst., Dept. of Civ. and San. Eng., Massachusetts Inst. of Technology. Refers to H. K. Barrows, G. E. Russell, C. M. Spofford.

ROOT, LLOYD EUGENE, Baltimore, Md. (Age 23.) Refers to T. F. Comber, Jr., T. F. Hubbard, F. W. Medaugh, J. T. Thompson, G. W. C. Whiting.

SKAU, GORDON GEORGE, Brooklyn, N. Y. (Age 28.) Jun. Engr., Board of Transportation, New York City. Refers to W. Beattie, L. E. Robbe.

SKERRETT-LANDRON, RICARDO, Santurce, Puerto Rico. (Age 57.) Asst. Chf. of Div., Dept. of the Interior, Govt. of Puerto Rico. Refers to M. V. Domenech, M. Font, F. Fortuno-Selles, A. S. Lucchetti-Otero, E. Ortega-Rosado, F. Pons, A. Rodriguez, E. Totti y Torres, C. del Valle Zeno.

SMEDBERG, NELS HAROLD, Seattle, Wash. (Age 22.) Refers to I. L. Collier, G. E. Hawthorn, C. C. May, C. C. More, R. G. Tyler.

SMITH, HERMAN MORSE, Greensboro, N. C. (Age 44.) Field Engr., Nashville (Tenn.) Bridge Co. Refers to A. J. Dyer, J. W. Elliott, B. O. Hood, H. G. Hunter, S. R. Webb.

SMITH, ROBERT ALEXANDER, Cranford, N. J. (Age 60.) Surveyor and Cons. Engr. Refers to L. L. Coudert, P. Guise, J. H. Henderson, J. T. Hopkins, N. D. Wortendyke.

STILSON, ALDEN EARL, New York City. (Age 32.) Chf. Engr., Morse Boulger Destructor Co. Refers to W. G. Bank, W. Beattie, C. E. Boesch, H. B. Cleveland, T. M. Curry, Jr., C. A. Emerson, Jr., I. S. Osborn, A. H. Place, D. W. Townsend.

TAYLOR, OLIVER GUY, Chevy Chase, Md. (Age 49.) Asst. Chf. Engr., National Park Service. Refers to W. M. Austin, W. M. Beaman, C. H. Birdseye, A. Jones, F. A. Kittredge, W. M. Robinson, Jr., H. J. Spelman, M. R. Tillotson.

TORRE, MARIO DE LA, Cali, Colombia. (Age 23.) Refers to V. R. Irvine, A. N. Johnson, S. Ospina, R. H. Skelton, S. S. Steinberg.

TROXELL, JOHN ADRON, Bellingham, Wash. (Age 22.) Refers to I. L. Collier, C. C. May, C. C. More, R. G. Tyler.

WILLIAMS, JAMES PATRICK, Yonkers, N. Y. (Age 35.) Sales and Service Engr., Alpha Portland Cement Co. Refers to E. K. Borchard, H. F. Finck, C. R. Hulsart, R. Radbill, F. J. Wright.

FOR TRANSFER

FROM THE GRADE OF ASSOCIATE MEMBER

ALLEN, HAROLD JOSEPH, Assoc. M., Detroit, Mich. (Elected May 25, 1931.) (Age 38.) Jun. Engr., U. S. Engr. Office. Refers to J. E. Bebb, G. W. Carlton, J. H. Cissel, J. W. Cowper, C. Y. Dixon, J. J. Dunkel, P. A. Fellows, O. H. S. Koch, D. A. Molitor, R. S. Swinton, J. A. Van den Broek.

NUGENT, HARRY WARFIELD, Assoc. M., New Madrid, Mo. (Elected Dec. 5, 1927.) (Age 35.) Vice-Pres. and Gen. Mgr., Wilbanks & Pierce, Inc., Dredging Contrs. Refers to J. S. Allen, D. A. MacCrea, W. A. K. Parkin, R. C. Pierce, A. M. Shaw, J. R. Wilbanks.

PAFFRATH, ERNST, Assoc. M., St. Louis, Mo. (Elected April 18, 1927.) (Age 46.) Asst. Sewer Commr. Refers to R. B. Brooks, H. E. Frech, E. R. Kinsey, A. Lewald, W. E. Rolfe, C. W. S. Sammelman, J. M. Slater, C. E. Smith, C. M. Talbert.

RAMIREZ, REINALDO, Assoc. M., San Juan, Puerto Rico. (Elected Sept. 10, 1923.) (Age 41.) Chf. Asst. Engr., M. of W. and Structure Dept., American R. R. Co. Refers to F. A. Barnes, R. R. Casellas, M. V. Domenech, M. Font, R. A. Gonzalez, E. S. Jimenez, W. W. Schlecht, R. M. Snell, E. Totti y Torres, C. del Valle Zeno.

FROM THE GRADE OF JUNIOR

ALBRIGHT, RALPH JOHN, Jun., Allentown, Pa. (Elected Oct. 14, 1929.) Age 32.) With McClintic-Marshall Corporation, Bethlehem, Pa. Refers to E. F. Ball, S. A. Becker, E. L. Durkee, C. D. Jensen, H. B. Thorn.

ARTHUR, LYNN JENNINGS, Jun., Indianapolis, Ind. (Elected July 16, 1928.) (Age 32.) Chf. Designer with Jeup & Moore, Inc. Refers to C. Brossman, H. O. Garman, W. K. Hatt, W. A. Knapp, J. W. Moore, R. B. Moore, R. B. Wiley.

BACKSTON, PHILIP, Jun., St. George, N. Y. (Elected Oct. 1, 1928.) (Age 27.) Sr. Engr., Dept. of Public Works, Div. of Engrs., State of New York. Refers to E. K. Hiles, L. T. Howard, E. R. Knight, H. K. Preston, R. W. Thoroughgood.

LUBKE, WALTER FRIEDRICH, Jun., North Tarrytown, N. Y. (Elected April 12, 1926.) (Age 32.) Constr. Mgr. with Louis Kell, Gen. Contr., Ossining, N. Y. Refers to W. E. Jessup, A. H. Krone, H. T. Larsen, R. C. Sandlass, G. A. Wieman, W. Wilson.

McGREW, EDWARD JOSEPHUS, Jr., Jun., New York City. (Elected July 11, 1927.) (Age 30.) Engr., Swart, Brent & Co., Inc. Refers to R. W. Baily, W. H. Bosier, J. E. Deignan, G. Dunn, C. Haydock, E. R. Needles, J. A. L. Waddell.

ROSENBERG, HERMAN HARRIS, Jun., Uniontown, Pa. (Elected Aug. 27, 1928.) (Age 32.) Engr., with David Goldberg (Uniontown Iron Works). Refers to F. M. McCullough, R. R. Schweitzer, C. B. Stanton, W. O. White, E. L. Zearley.

VAN GOENS, EDWIN, Jun., Los Angeles, Cal. (Elected Oct. 1, 1926.) (Age 32.) Jun. Civ. Engr., North Outfall Sewer Dept., and San Sewer Dept. Refers to F. A. Batty, W. C. Hogoboom, L. C. Larson, L. C. Mayer, F. W. Parsons, H. A. Van Norman, A. P. von Deesten.

The Board of Direction will consider the applications in this list not less than thirty days after the date of issue.